PB-242 914

MULTI-PURPOSE COMBINED SEWER OVERFLOW TREATMENT FACILITY, MOUNT CLEMENS, MICHIGAN

Vijaysinh U. Mahida, et al

Mount Clemens

Prepared for:

National Environmental Research Center

May 1975

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192076

PB 242 914

EPA-670/2-75-010
May 1975

Environmental Protection Technology Series

MULTI-PURPOSE COMBINED SEWER OVERFLOW TREATMENT FACILITY, MOUNT CLEMENS, MICHIGAN

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TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

1. REPORT NO.
EPA-670/2-75-010

2.

3. RECIPIENT'S ACCESSION NO.

4. TITLE AND SUBTITLE
MULTI-PURPOSE COMBINED SEWER OVERFLOW TREATMENT FACILITY, MOUNT CLEMENS, MICHIGAN

5. REPORT DATE
May 1975; Issuing Date

6. PERFORMING ORGANIZATION CODE

7. AUTHOR(S)
Vijaysinh U. Mahida and Frank J. DeDecker

8. PERFORMING ORGANIZATION REPORT NO.

9. PERFORMING ORGANIZATION NAME AND ADDRESS
City of Mount Clemens
1 Crocker Boulevard
Mount Clemens, Michigan 48043

10. PROGRAM ELEMENT NO.
1BB034; ROAP 21-ASY; TASK 149

11. ~~CONTRACT/~~GRANT NO.
11023 FAR

12. SPONSORING AGENCY NAME AND ADDRESS
National Environmental Research Center
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

13. TYPE OF REPORT AND PERIOD COVERED
Final

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

16. ABSTRACT

Combined sewer overflows from 212 acres within the City of Mount Clemens were conveyed to a treatment-park site. The overflows received initial treatment (settling and surface aeration) in a retention basin. Further treatment consisted of microstraining, disinfection, surface aeration in a series of lakelets, and filtration. The annual existing overflow of 2180 cu ft/acre-inch of rainfall had SS of 50 lbs/acre-inch and BOD_5 of 20 lbs/acre-inch. Treatment reduced the annual pollution load by 90 percent. The final lake sampling data has demonstrated that all water quality parameters for fishing, boating, and/or lawn sprinkling--except the toxic and deleterious substances parameters, which were not studied--were met. Very limited investigations were undertaken in the area of recreation, open space, and transitional land use. Treatment of combined sewer overflows was found to be more cost-effective than separation of an existing combined sewer system. This report was submitted in fulfillment of Project No. 11023 FAR by the City of Mount Clemens, Michigan, under the partial sponsorship of the U.S. Environmental Protection Agency. Work was completed as of August 1973.

PRICES SUBJECT TO CHANGE

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Cost comparison, Water quality, *Filtration, Sewers, *Chlorine, *Surface water runoff, *Water supply, *Water conservation, *Overflows, *Recreational facilities, Contaminants, Standards, Storage, *Disinfection, *Chlorination, *Combined sewers	Water pollution control, *Microstraining, *Combined sewer overflow, *Storm runoff, *Pollutants removal, *BOD removal, *Wastewater treatment, Urban water management, Multi-purpose treatment, Chlorine dioxide, Mount Clemens (Michigan), Stormwater reuse, Cost-benefit analysis, *Rainfall-runoff relationships	13B

18. DISTRIBUTION STATEMENT
RELEASE TO PUBLIC

19. SECURITY CLASS *(This Report)*
UNCLASSIFIED

20. SECURITY CLASS *(This page)*
UNCLASSIFIED

21. NO. OF PAGES
216

22. PRICE
$7.25

EPA Form 2220-1 (9-73)

☆ U. S. GOVERNMENT PRINTING OFFICE: 1975-657-593/5385 Region No. 5-11

EPA-670/2-75-010
May 1975

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MOUNT CLEMENS, MICHIGAN

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Program Element No. 1BB034

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FOREWORD

Man and his environment must be protected from the adverse effects of pesticides, radiation, noise and other forms of pollution, and the unwise management of solid waste. Efforts to protect the environment require a focus that recognizes the interplay between the components of our physical environment -- air, water, and land. The National Environmental Research Centers provide this multi-disciplinary focus through programs engaged in

- studies on the effect of environmental contaminants on man and the biosphere, and
- a search for ways to prevent contamination and to recycle valuable resources.

The "Multi-purpose Combined Sewer Overflow Treatment Facility Mount Clemens, Michigan" Demonstration Project brings into focus the beneficial uses of stormwater thereby attempting to meet the second objective as well as the intent of the Federal Water Pollution Control Act Amendments of 1972 (Public Law 92-500, 92nd Congress) under title II - Grants for Construction of Treatment Works, Section 201. (f) which states as follows:

> "The Administrator (of the Environmental Protection Agency) shall encourage waste treatment management which combines 'open space' and recreational considerations with such management."

A.W. Breidenbach, Ph.D.
Director
National Environmental
Research Center, Cincinnati

ABSTRACT

Combined sewer overflows from 212 acres within the City of Mount Clemens were conveyed to a treatment-park site. The overflows received initial treatment (settling and surface aeration) in a retention basin. Further treatment consisted of microstraining, disinfection, surface aeration in a series of lakelets, and filtration.

The annual existing overflow of 2180 cu ft/acre-inch of rainfall had SS of 50 lbs/acre-inch and BOD_5 of 20 lbs/acre-inch. Treatment reduced the annual pollution load by 90 percent.

The final lake sampling data has demonstrated that all water quality parameters for fishing, boating, and/or lawn sprinkling -- except the toxic and deleterious substances parameters, which were not studied -- were met. Very limited investigations were undertaken in the area of recreation, open space, and transitional land use.

Treatment of combined sewer overflows was found to be more cost-effective than separation of an existing combined sewer system.

This report was submitted in fulfillment of Project No. 11023 FAR by the City of Mount Clemens, Michigan, under the partial sponsorship of the U.S. Environmental Protection Agency. Work was completed as of August 1973.

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ACKNOWLEDGEMENTS

The authors wish to gratefully acknowledge the foresight and willingness of the City Manager, Edwin Whedon, P.E., the Mayor and the Commissioners of the City of Mount Clemens, to sponsor the Demonstration Project. The assistance and cooperation of the Superintendent of the Wastewater Treatment Plant, James Van Havermaat, P.E., in conscientiously operating the facility is also cordially acknowledged.

The authors also wish to acknowledge the active support of the U.S. Environmental Protection Agency's Project Officers Laurence B. O'Leary, P.E., Robert M. Buckley, P.E., and Richard Field, P.E.

Further, the authors wish to thank William H. Rosenkranz, P.E. Chief, and Francis Condon, P.E. Project Officer of the Municipal Technology Branch of the Environmental Protection Agency, Washington, D.C. for their continued interest and guidance as well as the staff of the Michigan Water Resources Commission for their encouragement and cooperation in processing this project and its subsequent study.

The authors are grateful to the Michigan Section of the American Consulting Engineers Council (ACEC) for having awarded this Demonstration Project the 'Grand Conceptor' engineering excellence award, and to the ACEC/USA for having awarded the 'Honor Award' during the ACEC 1971 Engineering Excellence Awards competition.

The authors are appreciative of the efforts of Ms. Candice Meyer for the careful preparation and review of the manuscript.

SECTION I

CONCLUSIONS

GENERAL CONCLUSIONS

1. The concept of collection and treatment of combined sewer overflows is a feasible, more efficient, and a better cost effective alternative than separation of an existing combined sewer system.

   The collection and treatment of overflows from the Mount Clemens study area of 212 acres reduced the annual pollution load as follows:

   a) 94 to 96 percent of removal of suspended solids (SS)

   b) 92 to 96 percent removal of BOD_5

   If separation of the combined sewer system had been undertaken, the annual pollutional load would have been reduced as follows:

   a) 35 percent removal of SS

   b) 88 percent removal of BOD_5

   The annual cost of collection and treatment of combined sewer overflows from the study area was $100,000 whereas the annual cost of separation of the combined sewer system would have been $185,000.

2. The treatment-park concept provides for the acquisition of land for use as a combined sewer overflow treatment facility with incidental use for open space, recreation and transitional land use area.

   The treatment-park concept seems most feasible because of the multipurpose, cost-effective use of public funds.

TECHNICAL CONCLUSIONS - General

1. The average annual rainfall of 30 inches in the 212 acres combined sewer service area would produce an annual overflow volume of 13,852,000 cu ft, assuming 60 percent of the rainfall is expected to go through the sewer system. The remaining 40 percent of the rainfall would either percolate into the ground, evaporate, or produce runoff directly to the Clinton River. The annual overflow volume contributed by the combined sewer system can be expressed as 2180 cu ft/acre-inch of rainfall.

2. The pre-construction studies indicated that the annual overflows would contain 302,500 lbs of SS and 121,000 lbs of BOD_5. These values can be expressed as 1427 lbs/acre/year or 50 lbs/acre-inch of rainfall for SS and as 570 lbs/acre/year or 20 lbs/acre-inch of rainfall for BOD_5. These loadings are based on overflow quality having an average SS of 350 mg/l and BOD_5 of 140 mg/l. It should be noted that the pre-construction data is composed of "dry year" data.

3. The post-construction studies indicate that the combined sewer overflow collection and treatment facility would reduce the annual pollution to the receiving stream to 12,100 lbs of SS and 5,200 lbs of BOD_5. These values can be expressed as 60 lbs/acre/year or 2 lbs/acre-inch of rainfall for SS and 25 lbs/acre/year or 1 lb/acre-inch of rainfall for BOD_5. These loadings are based on the treated overflow quality having a flow weighted average SS of 239 mg/l with a 95 percent confidence level of ±83 mg/l and an average BOD_5 mg/l of 75 mg/l with a 95 percent confidence level of ±25 mg/l.

4. The treatment facility final effluent had a flow weighted average SS of 14 mg/l with a 95 percent confidence level of ±2 mg/l, and an average BOD_5 of 6 mg/l with a 95 percent confidence level of ± mg/l.

5. The post-construction studies indicate that there is a 94 to 96 percent reduction in SS and a 92 to 96 percent reduction in BOD_5, depending upon whether the pre-construction or post-construction period overflow quality is used in the computations.

6. The Mount Clemens treatment concept evaluation indicates that it is a feasible and reliable concept. Lakelet No. 3 sampling data (Table 12) has demonstrated the capability of the treatment concept to acceptably renovate combined sewer overflows for fishing and boating and for lawn sprinkling. All water quality parameters, except the toxic and deleterious substances parameter (not studied), were met.

7. Comparison of the annual cost of collection and treatment facility for combined sewer overflows ($100,000) and the annual cost for separation ($185,000) within the 212 acres service area suggests there is annual savings of $85,000 or $400 per acre of service area. The cost of the collection and treatment facility (total project cost) for combined sewer overflows was $758,000 or $3,600 per acre of service area. Assuming a 30 year, 6 percent bond issue, the annual retirement cost would have been $55,000. The annual operation and maintenance cost was estimated at $45,000. A separation program by constructing new storm sewers would have cost $2,544,000, or $12,000 per acre. Separation by constructing new sanitary sewers would have cost approximately $24,000 per acre. Assuming a 30 year, 6 percent bond issue, the annual retirement cost for constructing storm sewers would have been $185,000. All costs are based on 1973 ENR Construction Cost Index of 1800.

   The city-wide collection and treatment facility (presently under construction) to serve a combined sewer area of 1471 acres is expected to cost $15,000,000 or $10,000 per acre of service area. Assuming a 30 year, 6 percent bond issue, the annual retirement cost would be $1,090,000. The annual operation and maintenance cost is estimated to be $171,000 ($171 per million gallons or $1.28 per 1000 cu ft). Of this cost, about 80 percent is attributable to the Retention Basin. Separation would have cost approximately $22,000,000 or $15,000 per acre. The higher separation costs are due to construction adversities expected in "downtown" business areas. All costs are based on 1973 ENR Construction Cost Index of 1800.

TECHNICAL CONCLUSIONS - Operation

Note: The following conclusions are the outcome of operating the EPA Demonstration Project.

1. The combined sewer overflow rate and volume monitoring methods used during the pre-construction studies (utilizing magnetic flow meters) did not perform satisfactorily. The flow meters were not capable of handling highly varying surges encountered.

2. The combined sewer overflow samplers did not perform satisfactorily. Large particulate matter, pieces of cloth, and debris blocked, at times, the sample tubes.

   The samplers were designed to operate during periods of overflow when the rate exceeded 0.5 cfs. This eliminated a certain number of samples during low intensity storms.

3. The U-trough and drop manhole method for intercepting combined sewer overflows operated satisfactorily and efficiently.

4. The combined sewage overflow pump station had electrode controls, which had to be kept clean for proper pump actuation. This became a regular maintenance procedure.

5. Winter operation of the floating aerators proved to be a difficult problem. The icing up of the aerator motor made the aerator unstable. When this situation was permitted to continue, the aerator soon would become exceedingly top heavy and would capsize.

   Summer operation of the aerators presented only one problem. The aerators in Lakelet No. 1 (Retention Basin) would tend to collect pieces of cloth, weeds, and other such foreign material and wrap it around either the impeller or the erosion plate which resulted in restricting the intake. This would put an extra load on the motor which would sometimes overload the aerator enough to shut it down.

6. If Lakelet No. 1 (Retention Basin) were to be operated on a long term basis, some method of sludge removal would be required. At the end of the one year study period, an average 9 inches of sludge accumulation was observed on the bottom of the lakelet. This amounts to 28,144 cu ft or 133 cu ft/acre/year. The Demonstration Facility was not intended to incorporate sludge removal.

7. The periodic operation of Lakelet No. 1 (sedimentation, aeration, etc.) and the occasional occurrence of oils in the combined sewer overflows indicated that the microstrainer is not suited in the Mount Clemens treatment concept.

8. Chlorination studies were initially proposed to be run for the purpose of evaluating chlorine dioxide as a better alternative for treating combined sewer overflows as compared to standard chlorine gas disinfection. Chlorine gas studies were run and a sufficient quantity of data was collected pertaining to dosing rates, costs and amounts of residual chlorine achieved. However, due to a lack of rainfall during the study period, a sufficient quantity of data was not collected for the chlorine dioxide studies to allow for any conclusive evaluation of the two alternatives. The future city-wide project proposed the use of chlorine dioxide as a method of disinfection such that further studies can be performed.

9. The sand filter functioned very well during the entire study period. The most noteworthy factor concerning the application of a sand filter for this project was that the unit could be discontinued from service for any period of time and then restarted with no special maintenance procedures being required. Such is not the case, for example, with a microstrainer which requires extensive cleaning of the screens when it is taken out of service.

10. Algae was present in the lake system in excessive quantities during the study year. It is our conclusion that phosphorus removal be incorporated in the Mount Clemens treatment-park concept.

11. Turbidity was observed in the lakes system. The lakes system was lined with clay; stone lining was deleted for cost saving. It is our assertion that stone lining is necessary to reduce turbidity.

12. During the one year study period less than 20 percent of the storms occurred during a regular, 8 hour day shift. This indicated that automation of the treatment facility is a necessity. Automation less than 100 percent would have required a 3 shift per day type of operation.

    Automation of the treatment facility was estimated to cost $40,000. If the facility would not have been automated, the personnel costs would have been $62,900 instead of $32,900.

13. The annual cost of operation and maintenance during the study year was $45,000. This cost includes only the operation of the facility, as no park facility was developed at the time. This project was operated by city personnel from the adjacent sewage treatment plant. This situation provided operating personnel for the project as they were needed, but one operator was assigned full time.

    The city-wide project (under construction) for treatment of combined sewer overflows is based on the city conveying its dry weather flow sanitary sewage to the regional system. The combined sewer overflow facility would be staffed by full time personnel.

TECHNICAL CONCLUSIONS - Design Procedures

Note: The following conclusions are the outcome of experience gained during the design of the EPA Demonstration Project and the city-wide project.

The Demonstration Project had 3 lakelets, an aerated Lakelet No. 1 acting as a retention basin, Lakelet No. 2, and an aerated Lakelet No. 3. The total minimum detention time was 16 days and the total maximum detention time was 19 days, at 1 mgd flow rate. The City-Wide Project has an aerated retention basin and 3 lakelets – an aerated Lakelet No. 1, partially aerated Lakelet No. 2, and Lakelet No. 3. Prior to aeration and retention the storm flows will go through a series of three sedimentation resuspension chambers (SRC). The total minimum detention time proposed is 8.6 days and the maximum detention time is 12.4 days, at 4 mgd flow rate. The final effluent from the lakes system is expected to be as good as in the Demonstration Project, in spite of increasing the flow rate from 1 to 4 mgd, due to the additional aeration provided.

The total aeration provided in the Demonstration Project was for 10.8 days. The total aeration provided in the city-wide project is for 9.5 days minimum and 11.3 days maximum. It should be noted that the proposed aeration in Lakelet No. 1 would reduce BOD_5 from 50 mg/1 to 10 mg/1.

1. An important part of the design process is an analysis of the rainfall-runoff relationship. The frequency and intensity of storms, basin characteristics, design storm, and occurrences of additional storms before the design storm is dewatered, are some of the considerations in the analysis.

2. An existing sewer system investigation is necessary to produce information concerning the actual watershed service area, the degree of separation within the area, the extent of deterioration of the sewer structures, and the probable capacities within the combined sewer system. This information is crucial in the detailed design of interceptors and retention and treatment facilities.

3. The U-trough and drop manhole method for intercepting combined sewer overflows is feasible and efficient. By constructing the U-trough and drop manhole such that the overflows would be intercepted ahead of the existing diversion chambers, the diversion chambers can be left alone to divert the dry weather flow and remain as an emergency overflow outlet in case of a combined sewer overflow pump station failure.

   The U-trough should be sized to carry only the desired peak dry weather flow in order to minimize the surcharge to be conveyed to the dry weather flow disposal system. An adjustable weir on the U-trough permits control of the surcharge, and the weir height can be easily field adjusted.

   The hydraulic gradient of the combined sewer overflow interceptor should be below the top edge of the U-trough. Thus only the maximum capacity of the U-trough would be conveyed to the dry weather flow disposal system.

4. Direct measurement of rate and volume of overflows pumped into the retention basin is better than an indirect measurement (monitoring the water level in the basin and recording pumpage out).

5. The flow proportioned sampler used during the Demonstration Project was improved upon in the city-wide project by providing 3 composite sample jars. The sampler deposits samples collected during the first period (adjustable, 0-4 hours) in the first sample jar. The sampler deposits samples collected during the next period (adjustable, 0-12 hours) in the second jar. The sampler deposits the rest of the samples in the third jar.

   The sampling pump in the city-wide project is provided with a macerating unit which macerates the combined sewage and reduces organic and inorganic solids to a size that enables them to be pumped by the sample pump. The macerating unit has the capability of being rotated counterclockwise to permit unclogging.

6. The sedimentation-resuspension chambers, proposed under the city-wide project, were designed to provide a minumum of 30 minutes of settling time. The peak overflow design rate is 0.0058 fps (3760 gpd/sq ft or 1.77 mm/sec). Assuming specific gravity of 1.2, the peak rate is expected to settle solids 0.15 mm and greater. Thus the SRC have the design capability to settle out nearly 90 percent of settleable solids.

   Upon cessation of any inflow, the settled material is to be resuspended by means of a series of dual ejector units which utilizes forced air and high pressure water jets to create a turbulent mixing effect.

   The contents of the SRC can then be conveyed directly to the dry weather flow disposal system.

7. Depending upon whether the Mount Clemens treatment concept or the treatment-park concept is used, sand filters may or may not be needed. If sand filters are needed, they should be placed between the clarifier and the flow-through-type first lakelet.

SECTION II

RECOMMENDATIONS

GENERAL RECOMMENDATIONS

1. The alternative of collection and treatment of combined sewer overflows should be investigated as a viable alternative to separation of an existing combined sewer system because it is a feasible, more efficient, and a better cost-effective alternative.

2. General application of the treatment-park concept, as proposed in Mount Clemens, is recommended because of the multi-purpose and cost-effective use of public funds.

3. The combined sewer overflows should initially be conveyed to sedimentation-resuspension chambers (SRC) and then to an aerated retention basin. The settled material in the SRC should be resuspended -- preferably without the use of external water -- and discharged directly to the dry weather flow disposal system. The aerated retention basin should include a clarifier to remove phosphorus. The phosphorus-laden sludge should also be removed directly to the dry weather flow disposal system. The effluent from the clarifier can then be conveyed to a separately located lakes system for further treatment and multi-purpose uses.

TECHNICAL RECOMMENDATIONS

1. Tests should be conducted for toxic and deleterious substances in Lakelet No. 3. Such tests should confirm the acceptability of the use of the renovated overflows for recreation. (This program is proposed to be conducted by the City of Mount Clemens under its city-wide project.)

2. Continue chlorine dioxide studies for more conclusive information. (This program is proposed to be conducted by the City of Mount Clemens under its city-wide project.)

3. Fully automate the combined sewer overflow treatment facility. The operation and maintenance staff required for the facility can then be reduced to a regular 8-hour-day-shift basis.

4. Develop improved methods for monitoring existing combined sewer overflows.

5. Develop improved methods for preventing icing problems of floating aerators.

6. Review the Technical Conclusion (outlined in Section I) as a guide or check-list to develop the design for a specific combined sewer overflow project.

7. The combined sewer overflow interceptors should be designed for stormwater runoff using a 5 to 10 year storm recurrence interval. The extra capacity in the interceptor, over that which might be capable of delivery by the collection system, would be available as an outlet for the future installation of additional relief storm sewers.

8. For basins that are not complex and have generally 200 acres or less, it is recommended that the design storm runoff be analyzed by the Rational Method. For basins in excess of 200 acres, a modified Rational Method should be used.

9. In spite of 90 percent settling expected to occur in the SRC, there will most likely be some fine particulate material carried over into the retention basin. This material may settle in the retention basin and, upon the dewatering of the retention basin, produce an odoriferous condition. It is therefore deemed necessary to provide a flushing system for use in removing the settled material from the retention basin.

10. The capacity of the retention basin should be based on the analysis of rainfall records and mass diagrams. The in-system storage possibility provided by the combined sewer overflow interceptor should be considered in the design of the retention basin storage. The mass diagram assists in determining the most reasonable combination of retention basin capacity, dewatering rate, and the number of emergency overflows permitted from the retention basin.

    The determination of an appropriate dewatering rate should take into consideration an economical stormwater treatment facility (including a possible phosphorus removal process unit) rate in relation to a rate that would dewater the retention basin in sufficient time to provide retention capacity for the next rainstorm.

Following the determination of the most favorable capacity and dewatering rate for the retention basin, the final layout can then be designed, including the supplemental systems for aeration, disinfection, and flushing.

It should be noted that the primary function of the retention basin is to capture and retain combined sewer overflows. The treatment aspect of the retention facility is secondary. It behooves the design engineer to take maximum advantage of the detention time in a retention basin. The Demonstration Project aerated retention basin (Lakelet No.1) permitted 80 percent BOD_5 removal within 4 days of detention time. The provision of mechanical aeration in the receiving-retention basin -- as an extremely efficacious location for initiating the satisfaction of total BOD -- is considered, therefore, one of the more important features of the Mount Clemens concept. Under full retention basin conditions, it is anticipated that it would take about 8 days to completely dewater the retention basin. Even when only partially filled, the time is usually abundant to be conducive toward stabilization. The retention basin should be open to the atmosphere; this is not only more appropriate for maximum oxygen transfer, but is substantially less expensive than an enclosed basin. However, land availability and value must be considered in the determination of whether the basin can be left open or whether it must be enclosed to allow use of the land area above the basin.

11. All flows discharging directly to the river from the retention basin in Mount Clemens are to be disinfected in a chlorination basin prior to discharge. The chlorination facility was incorporated as a means of meeting specific requirements set forth by the State of Michigan. However, the true cost effectiveness of incorporating chlorination facilities at a retention basin site is worthy of further study and evaluation considering: (1) the high degree of sedimentation which is expected in the SRC and retention basin, (2) the aeration of the overflows in the retention basin during the filling period and (3) the frequency of the overflows from the retention basin to the receiving stream.

12. Algae was present in the lake system in excessive quantities during the Demonstration Project studies. A phosphorus removal mechanism must be incorporated in the treatment of the overflows at some location between the retention basin and the lake system. A clarifier, receiving flows from the retention basin, would permit phosphorus removal. The sludge from the clarifier could be discharged to the dry weather flow disposal system.

    The city-wide project for Mount Clemens contemplates the use of the existing clarifiers for potential phosphorus removal. Moreover, it is planned to investigate the use of various substances -- including the use of water supply filtration plant waste sludge (another current disposal problem) -- for phosphorus removal in order to economically remove enough phosphorus from the process water to reduce the algae growth in the recreation lake.

13. A flow-through type, three lake system should be provided for the final treatment of combined sewer overflows. The first lake should be isolated and provided with submerged aeration. The submerged aeration would eliminate winter maintenance problems, maximize the treatment efficiency, and help produce a high quality effluent that would not be detrimental to fishing and boating activities. The second lake would act as a transition and polishing lake. The third lake would be used for limited recreational activities. Submerged aeration may be advisable in the third lake to insure that a high dissolved oxygen concentration would be maintained. Submerged aeration is also safer if people were to use the lake.

    Based on our experience with combined sewer overflows and with aerated flow-through lagoons, we would suggest that a lake system be designed to provide 7 to 10 days detention time and have a depth of about 10 feet.

SECTION III

INTRODUCTION

BACKGROUND

In a letter dated September 30, 1966, the "Division of Facilities Demonstration Grants", of the "Federal Water Pollution Control Administration", now a part of the United States Environmental Protection Agency, put forth an appeal to consulting engineers throughout the country to utilize the available grants to "demonstrate and evaluate new or improved methods of controlling pollution from combined sewers or storm water discharges." As municipal planning and engineering consultants, Spalding, DeDecker & Associates, Inc. had been seriously studying the problem of combined sewer overflows prior to receiving the aforementioned letter. Hence, on October 14, 1966, the firm submitted an application to demonstrate a new concept for the treatment of combined sewer overflows. It had occurred to Spalding-DeDecker that the multiplicity of Federal Grants-in-aid programs (such as "Open Space," "Recreation," "Water Pollution Abatement" and "Air Pollution Neutralization") aimed at curing some of the nation's urban ills, could be combined into a single project which could fulfill the hopes of all such programs and incidentally result in a conservation oriented model viewed as the prototype for a more economical solution of the combined sewer overflow problem. Moreover, water in the form of ponds, or lakelets, is a universally welcomed amenity for any park or open-space setting.

The City of Mount Clemens, being a typical American City of approximately 20,000 residents and having a combined sewer system, was solicited as a sponsor for the demonstration of the concept as a means of solving its combined sewer overflow problem. Subsequently on June 7, 1967, the City of Mount Clemens passed a resolution to authorize the filing of an application for a Facilities Demonstration Grant to study the Multi-purposed Combined Sewer Overflow Treatment Facility concept.

In the meantime, the State of Michigan Water Resources Commission was in the process of citing the City of Mount Clemens for pollution of the Clinton River which culminated in the execution, on November 16, 1967, of a "Stipulation", or agreement, between the City and the Water Resources Commission. The "Stipulation" was concerned with both dry-weather flows and wet-weather flows; however, this report is concerned only with the wet-weather flow problem, i.e., combined sewer overflows.

With regard to combined sewer overflows, the "Stipulation" called for the submission of a report on or before June 1, 1968, construction plans and specifications by January 1, 1970, and construction to be completed by June 1, 1972. Work done in fulfillment of the "Stipulation" has to date resulted in the development of this report and the initiation of construction of a City-wide Combined Sewer Overflow Renovation Project as described later in this report.

PURPOSE AND SCOPE

The purpose of this project was to investigate and evaluate the concept of treating combined sewer overflows in a city treatment-park facility capable of providing recreational benefits, open space, and providing a buffer or transitional zone between residential, commercial, and industrial areas of a city as a more desirable alternative to a sewer separation program. Figure 1 illustrates this general concept.

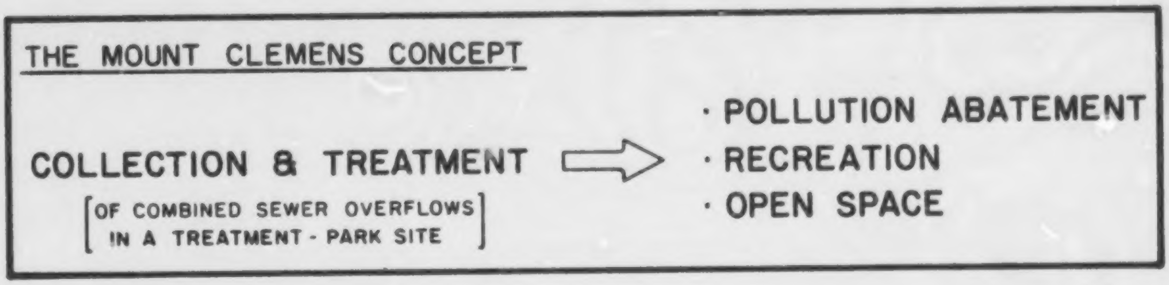

THE MOUNT CLEMENS CONCEPT
COLLECTION & TREATMENT
[OF COMBINED SEWER OVERFLOWS IN A TREATMENT - PARK SITE]
· POLLUTION ABATEMENT
· RECREATION
· OPEN SPACE

FIGURE 1. MOUNT CLEMENS CONCEPT FOR COMBINED SEWER OVERFLOW TREATMENT

The scope of the project was altered somewhat during the course of work in order to best achieve the main purpose, i.e., to demonstrate the feasibility of the Mount Clemens Concept. The actual investigations performed developed from the initially proposed specific objectives as described below:

1. "Collect data on the combined sewer overflows and on the affected reach of the Clinton River."
2. "Collect data on the rainfall intensities, frequencies of occurrence, runoff, and combined sewer overflows." This objective was met only in part. Specifically, during the design and construction of the demonstration facility, two sewer districts were monitored to ascertain the volume and pollutional content of the overflows which occurred during that time period. These studies supplied some baseline data for use during the treatment process evaluation.
3. "Design and construct a collection and treatment facility for the combined sewer overflows, as outlined in the concept." The entire City could not be included in the demonstration of the concept so only the two sewer districts nearest the project

site were studied. The two districts (the Spruce Street Drain and the Clemens Street Drain) served an area of 212 acres. This was considered to be sufficient for the demonstration of the concept feasibility.

4. "Operate the treatment facility to evaluate the performance of each process step, as well as the total process." The cost of operation and maintenance of the facility was studied for the purpose of extrapolating costs for any new facilities of this nature as well as to evaluate the total cost of this project as compared to a sewer separation program.

5. "Compare cost of collection and treatment of combined sewer overflows versus cost of a sewer separation program." A detailed sewer separation program and cost estimate was prepared for the service area of the project in order to compare the multi-purpose concept as an efficient and economical alternative for the abatement of the pollution problem caused by combined sewer overflows.

6. "Study the recreational benefits, the public reaction towards storm water renovation, and the effect of a city park-treatment facility as an open space and a transitional buffer zone." Only very limited investigations were undertaken in the area of recreation, open space, and transitional land use. The park facilities were not developed.

7. "Submit Project Report to the Environmental Protection Agency."

The Mount Clemens concept physically developed into a Treatment-Park Facility as illustrated in Figure 2.

In addition to the above described objectives, the design basis for the city-wide combined sewer overflow collection and treatment facility is presented with estimated operation and maintenance costs plus suggestions for applying the concept in other cities.

STUDY AREA

City of Mount Clemens

The City of Mount Clemens is located in Southeastern Michigan approximately 27 miles (center to center) northeast of the City of Detroit. This portion of the state is rapidly developing as an urban area. The Clinton River, which now receives all of the wastewater (treated dry-weather flow and untreated combined sewer overflows) from Mount Clemens, runs through the center of the City which is located approximately 6 miles upstream from the mouth of the river. Figure 3 shows the location of the City of Mount Clemens with respect to Detroit and the surrounding area. Figure 4 shows the location of the Clinton River Drainage Basin in the State of Michigan.

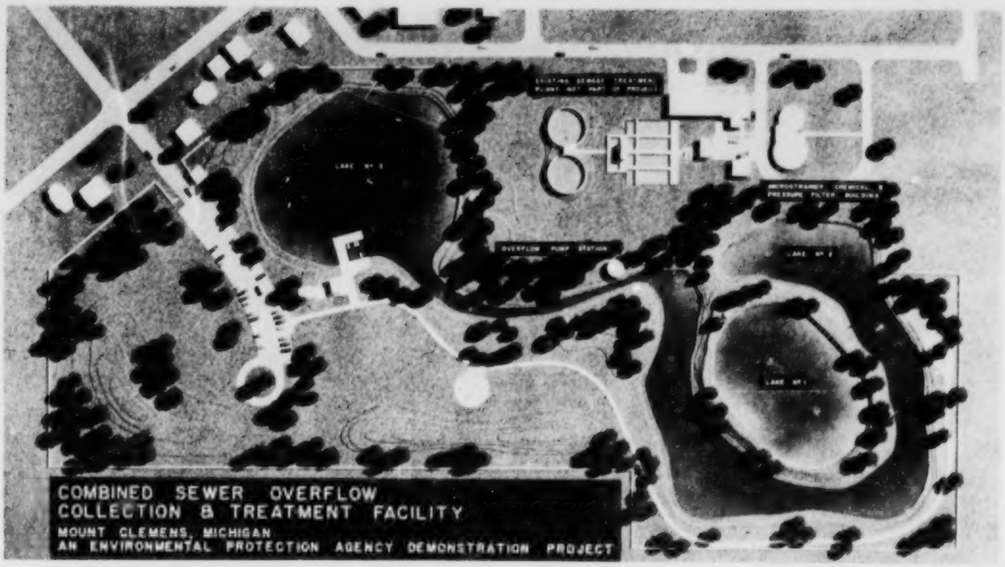

EXISTING SEWAGE TREATMENT PLANT NOT PART OF PROJECT
MICROSTRAINER CHEMICAL & PRESSURE FILTER BUILDING
OVERFLOW PUMP STATION
COMBINED SEWER OVERFLOW
COLLECTION & TREATMENT FACILITY
MOUNT CLEMENS, MICHIGAN
AN ENVIRONMENTAL PROTECTION AGENCY DEMONSTRATION PROJECT

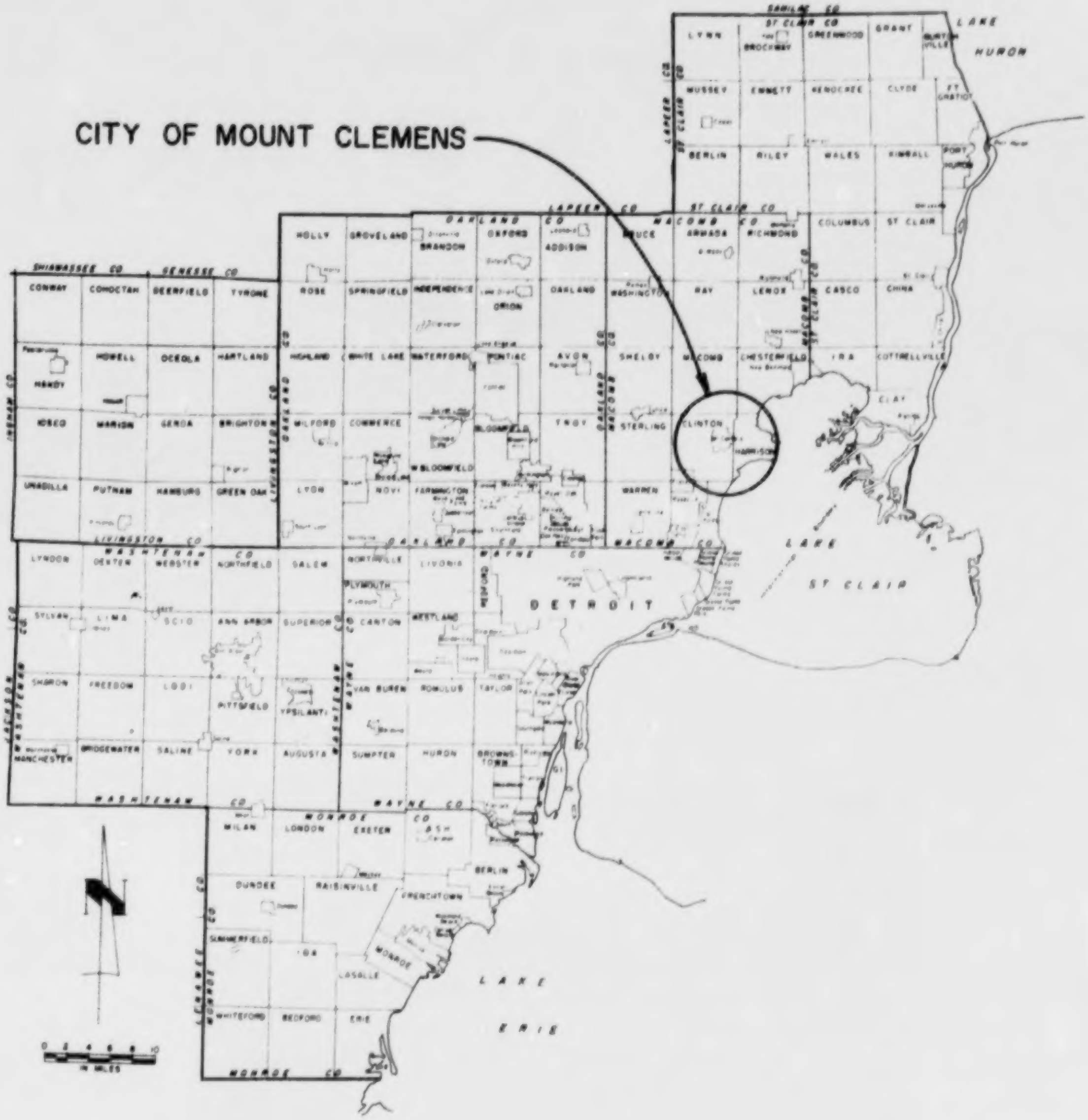
FIGURE 2. MOUNT CLEMENS PROPOSED TREATMENT-PARK FACILITY

The early stages of history in Mount Clemens began when it was incorporated as a village in 1837. There was a steady growth in commercial and residential development and in 1879 Mount Clemens changed from a village to a city status. All utilities, such as streets, water mains, and sewers were constructed as needed and kept pace with other growth in the City. For practical and economic reasons at the time, most of the sewers were built as a combined sewer system, discharging directly to the Clinton River.

Mount Clemens has now approached a stage of maturity with respect to population growth. The 1970 Federal Census revealed a population of 20,476, with the ultimate capacity of the land being about 26,000 people. The only major changes visible in the City would be the result of the demolition of existing older residential buildings in favor of construction of new multi-family and/or office buildings. Figure 5 shows a portion of the central business district with surrounding residential areas near the river.

The land uses within the City in 1965 were approximately 9% industrial, 5% commercial, 38% residential, with the remaining portions being public, transportation, and vacant areas. Residential and commercial developments are expected to increase these respective percentages of land used in the near future.

Mount Clemens covers an area of 3.58 square miles and has about 65 miles of sewers, 90 percent of which are of the combined type. The combined


CITY OF MOUNT CLEMENS

SANILAC CO
ST CLAIR CO
LAKE HURON
LYNN
BROCKWAY
GREENWOOD
GRANT
BURTCHVILLE
MUSSEY
EMMETT
KENOCKEE
CLYDE
FT GRATIOT
BERLIN
RILEY
WALES
KIMBALL
PORT HURON
LAPEER CO
ST CLAIR CO
MACOMB CO
OAKLAND CO
HOLLY
GROVELAND
BRANDON
OXFORD
ADDISON
BRUCE
ARMADA
RICHMOND
COLUMBUS
ST CLAIR
SHIAWASSEE CO
GENESEE CO
CONWAY
COHOCTAH
DEERFIELD
TYRONE
ROSE
SPRINGFIELD
INDEPENDENCE
ORION
OAKLAND
WASHINGTON
RAY
LENOX
CASCO
CHINA
HOWELL
OCEOLA
HARTLAND
HIGHLAND
WHITE LAKE
WATERFORD
PONTIAC
AVON
SHELBY
MACOMB
CHESTERFIELD
IRA
COTTRELLVILLE
HANDY
CLAY
IOSCO
MARION
GENOA
BRIGHTON
MILFORD
COMMERCE
BLOOMFIELD
TROY
STERLING
CLINTON
HARRISON
UNADILLA
PUTNAM
HAMBURG
GREEN OAK
LYON
W BLOOMFIELD
NOVI
FARMINGTON
WARREN
INGHAM CO
LIVINGSTON CO
LAKE ST CLAIR
WASHTENAW CO
WAYNE CO
LYNDON
DEXTER
WEBSTER
NORTHFIELD
SALEM
NORTHVILLE
LIVONIA
PLYMOUTH
DETROIT
SYLVAN
LIMA
SCIO
ANN ARBOR
SUPERIOR
CANTON
WESTLAND
SHARON
FREEDOM
LODI
PITTSFIELD
YPSILANTI
VAN BUREN
ROMULUS
TAYLOR
JACKSON CO
MANCHESTER
BRIDGEWATER
SALINE
YORK
AUGUSTA
SUMPTER
HURON
BROWNSTOWN
MONROE CO
MILAN
LONDON
EXETER
ASH
BERLIN
DUNDEE
RAISINVILLE
FRENCHTOWN
SUMMERFIELD
IDA
MONROE
LASALLE
LENAWEE CO
WHITEFORD
BEDFORD
ERIE
LAKE ERIE
N
0 2 4 6 8 10
IN MILES

FIGURE 3. REGIONAL LOCATION MAP

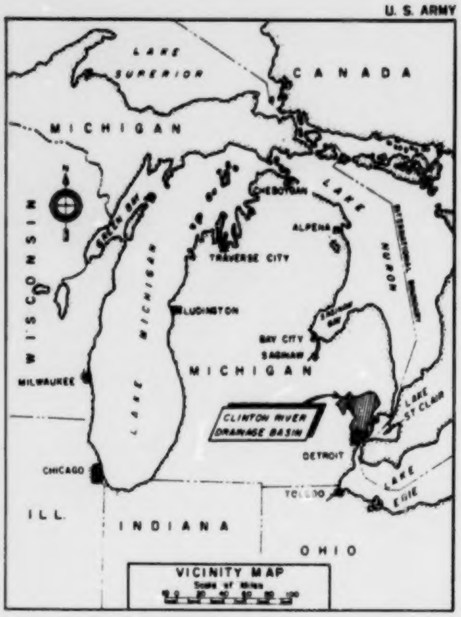

U. S. ARMY
LAKE SUPERIOR
CANADA
MICHIGAN
WISCONSIN
GREEN BAY
LAKE MICHIGAN
CHEBOYGAN
LAKE HURON
ALPENA
TRAVERSE CITY
LUDINGTON
SAGINAW BAY
BAY CITY
SAGINAW
MICHIGAN
MILWAUKEE
CLINTON RIVER DRAINAGE BASIN
LAKE ST CLAIR
DETROIT
CHICAGO
TOLEDO
LAKE ERIE
ILL.
INDIANA
OHIO
VICINITY MAP
Scale of Miles
10 0 20 40 60 80 100

FIGURE 4. VICINITY MAP

FIGURE 5. CENTRAL BUSINESS DISTRICT

sewage overflows occur at 24 locations within the City along a 3 mile reach of the Clinton River. Figure 6 shows the location of the Combined sewage overflow points within the City. The majority of the combined sewers within the City are capable of draining storm water at a design rate equivalent to a present day "two-year storm". The design flow/dwf ratio of the dry-weather-flow interceptors is approximately three times the dry weather flow.

The City of Mount Clemens supplies an average of 9.7 MGD of water to the city consumers which include some consumers in the surrounding townships. The City now purchases approximately 40% of this water supply from the City of Detroit while the balance is processed by the City's water filtration plant which obtains its water from Lake St. Clair.

The City's trickling filter wastewater treatment plant treats an average of 4.3 MGD. With the City's population of 20,476 (1970 Census) and the incidence of moderate industry, the estimated flow to the treatment plant would normally be expected to be 3 to 3.5 MGD, hence the surplus above this amount suggests a slightly higher than normal infiltration. A limited inspection of the sewer system has shown this to be the case.

The Clinton River

The Clinton River covers a distance of approximately 65 miles from its headwaters to the mouth. The basin is approximately 32 miles long and 36 miles wide, measured at the longest and widest parts. The total drainage area in the basin is approximately 760 square miles which includes portions of Macomb, St. Clair, and Oakland Counties.

The U.S. Geological Survey Gaging Station at Mount Clemens has been in service since 1934 and has recorded a maximum flow of 21,200 cfs with an average flow of 468 cfs. The augmented 7 day drought flow at Mount Clemens (10 year frequency), which includes the major point discharges within the Clinton River Basin, is 165 cfs.[1]

The Clinton River (from Pontiac to the mouth and the Red Run Basin) receive the discharges of treated effluent from 25 municipal and private sources, 15 of which contribute no significant oxygen consuming load. Typically the industries are engaged in general manufacturing, machinery operations, production of paper, and painting of steel parts. Wastes include toxic metals, soluble oil, BOD, solids, and heat from cooling water. However, many of these treatment plants are now being phased out of existence in favor of a regional wastewater collection and treatment system for sanitary sewage.

"The Clinton River has a limited number of water related recreational facilities. The reach between Mt. Clemens and the mouth has boat launching ramps and docking berths. The main reason for this lack of facilities is the fact that most of the stream is too small for boat traffic. Water pollution has also had a detrimental effect on boating activity. Pollution for most practical purposes, has eliminated fishing and swimming in the main stem of the Clinton River."[1]

Climate

"The climate of the City of Mount Clemens is somewhat modified by the Great Lakes which warm the air in the winter and cool it in the summer. This climate is typical of the entire lower Great Lakes area and can be generalized as having a wide seasonal temperature variation, many storms, and a relatively constant yearly precipitation distribution. In the winter, this precipitation is usually in the form of snow.

The mean yearly temperature is about 50°F, while the mean winter and summer temperatures are about 35°F and 65°F, respectively.

There is an average yearly precipitation of 30 inches within the Clinton River basin."[1]

References 1, 2, 3 and 4 contain detailed data from the subject area pertaining to flooding, water quality, physical conditions, etc., and should be referred to if further information is desired.

Project Site

Since the entire City could not be included in the Demonstration Project, the two sewer districts closest to the project site were chosen for the demonstration of the concept. The service areas for these two sewer districts covered 212 acres within the City, plus some partially developed areas outside of the City. These areas were predominently residential in nature, but limited commercial areas were also present.

The two sewer districts (the Clemens St. Drain and the Spruce St. Drain) were intercepted within the project site as described elsewhere in this report. The project site location is shown with respect to the City on Figure 6.

Figure 7 shows the two sewer service areas which were intercepted for the project. The Clemens Street Drain is a 29" x 51" box sewer servicing about 137 acres within the City. The Spruce Street Drain is a 42" sewer which serves an area of 75 acres within the City and conveys wastewater through the adjoining Clinton Township before re-entering the City prior to discharge into the Clinton River. The drainage area of the Spruce Street Drain which was outside the City limits had no substantial effect on this project.

Both the Clemens Street Drain and the Spruce Street Drain were initially designed as combined sewers discharging directly into the Clinton River. Then, after the trickling filter sewage treatment plant was constructed, the dry weather flows were intercepted and conveyed to the plant. Not until the construction of the Demonstration facility was the wet weather flow prevented from discharging to the river.

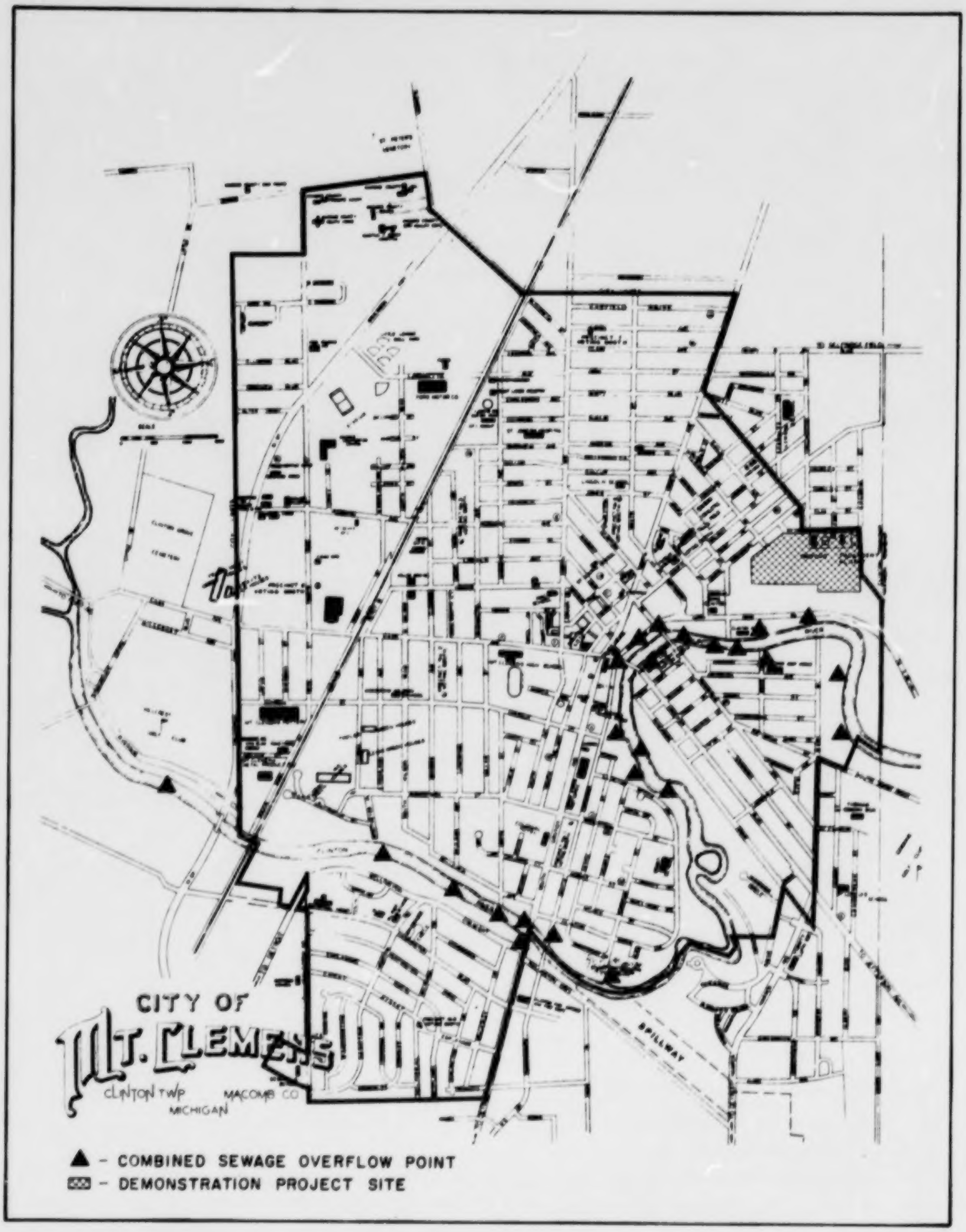

CITY OF
MT. CLEMENS
CLINTON TWP
MACOMB CO
MICHIGAN
CLINTON
SPILLWAY
▲ - COMBINED SEWAGE OVERFLOW POINT
▨ - DEMONSTRATION PROJECT SITE

FIGURE 6. COMBINED SEWER OVERFLOW POINT LOCATIONS

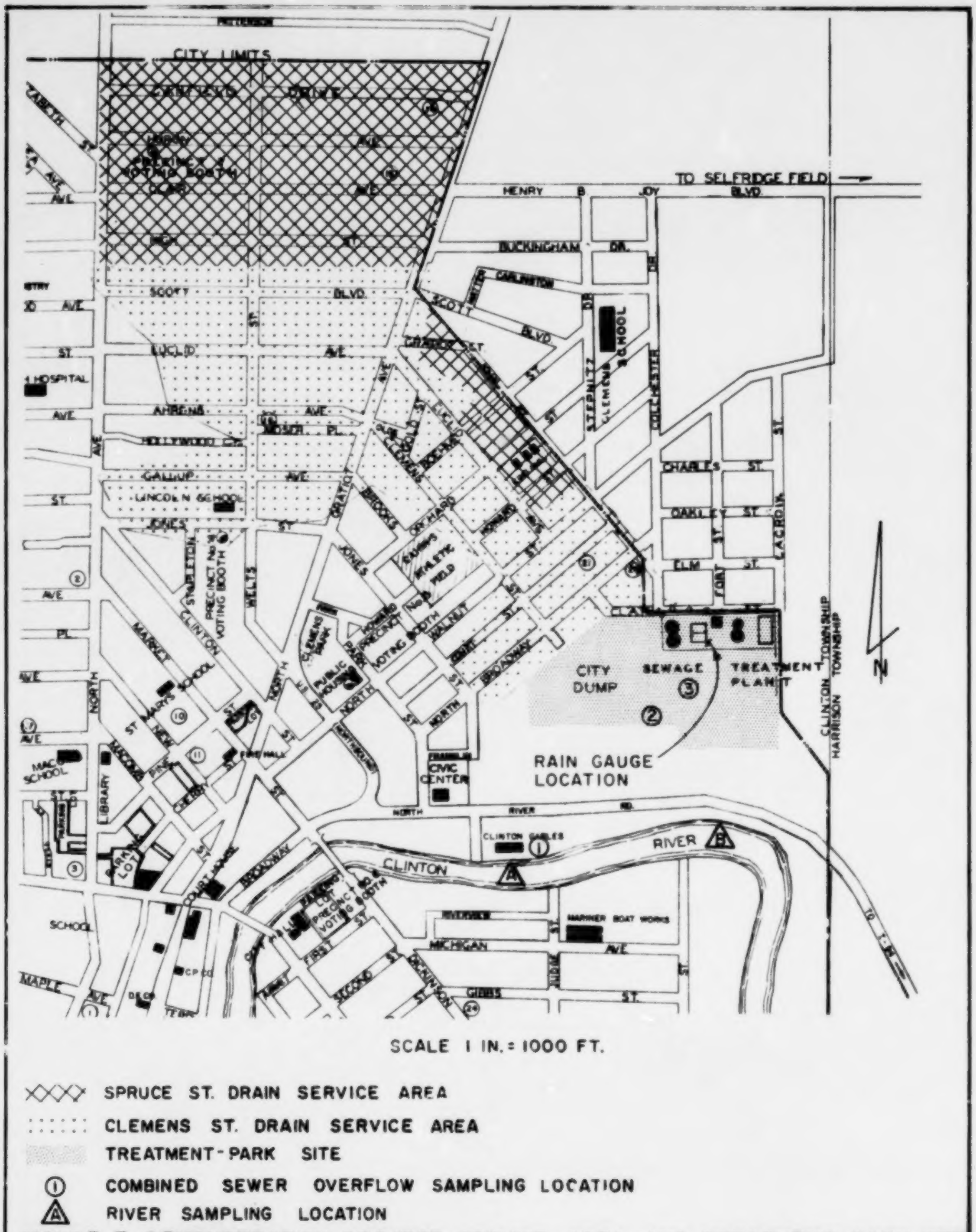

CITY LIMITS
TO SELFRIDGE FIELD
HENRY B JOY BLVD.
BUCKINGHAM DR.
SCOTT BLVD.
EUCLID AVE.
AHRENS AVE.
HOLLYWOOD CT.
GALLUP AVE.
LINCOLN SCHOOL
JONES ST.
CHARLES ST.
OAKLEY ST.
ELM ST.
CLEMENS SCHOOL
COLCHESTER
LACROIX
CLINTON TOWNSHIP
HARRISON TOWNSHIP
CITY DUMP
SEWAGE TREATMENT PLANT
RAIN GAUGE LOCATION
HOSPITAL
MAC SCHOOL
ST MARYS SCHOOL
LIBRARY
FIRE HALL
CIVIC CENTER
PUBLIC HOUSING
CLEMENS PARK
PARKING LOT
COURT HOUSE
CLINTON GABLES
CLINTON RIVER
MARINER BOAT WORKS
NORTH RIVER RD.
MICHIGAN AVE.
GIBBS ST.
MAPLE AVE.
BROADWAY
FIRST ST.
SECOND ST.
CLINTON
MARKET
NORTH
GRATIOT
SCHOOL
N
SCALE 1 IN. = 1000 FT.
SPRUCE ST. DRAIN SERVICE AREA
CLEMENS ST. DRAIN SERVICE AREA
TREATMENT-PARK SITE
COMBINED SEWER OVERFLOW SAMPLING LOCATION
RIVER SAMPLING LOCATION

FIGURE 7. DEMONSTRATION PROJECT SERVICE AREA AND TREATMENT-PARK SITE

Rain Gauging Station

The rainfall data utilized in this study was obtained from an official rain gauge station of the Southeast Michigan Council of Governments, otherwise known as SEMCOG. This rain gauge is a 12" Universal model and is operated according to National Weather Service guidelines. This particular rain gauge is part of a Southeast Michigan Network which consists of some 75 official rain gauges in the Detroit metropolitan area. The rain gauge data is processed monthly by the Michigan Weather Service office of the Michigan Department of Agriculture for SEMCOG. SEMCOG then distributes this information to all subscribing personnel or agencies in the Detroit metropolitan area.

The Mount Clemens rain gauge, officially referred to as station M-02, is located at the Mount Clemens Sewage Treatment Plant as shown in Figure 7. The operation of the Universal Rain Gauge, as pictured on location in Figure 8, is as follows. Rain or snow enters the rain gauge through a 12" cylindrical opening at the top of the rain gauge and is collected in a bucket. The weight of the bucket is continuously monitored and recorded through the use of a strip chart recorder. Thus, the recorded increase in weight of the bucket over any period of time can be readily converted to inches of rainfall.

The Michigan Weather Service office collects the rainfall record graphs monthly and from them derives tables illustrating precipitation per hour. The Michigan Weather Service office also records precipitation on a daily and monthly basis for each rain gauge and produces maps of the Detroit metropolitan area illustrating monthly total precipitations for each station at their locations. Finally, the Michigan Weather Service office produces an annual summary of precipitation for the Southeast Michigan Network which contains maps of the Detroit metropolitan area showing total annual precipitation for each station at their locations, maximum precipitation for various time intervals for each station, reasons for various rain gauge failures, and a brief report. The original rain gauge graphs are stored in the Michigan Weather Service office of the Michigan Department of Agriculture in East Lansing.

The ideal location for the rain gauge would have been a position near the center of the total service area. (The service area can be seen on Figure 7). However, the rain gauge at the treatment plant, being in operation for a long period of time and being adjacent to the service area, was considered sufficient for the purposes of this report. The distance of the farthest point within the service area from the rain gauge was 5700 feet.

There were no substantial differences between the rainfall characteristics of the Mount Clemens rain gauge station and the other rain gauge stations in the Southeast Michigan Network during the pre-construction period. Table 1 shows the average total annual precipitation for all of the reporting rain gauge stations in the Southeast Michigan Network. The Table also includes Mount Clemens' total annual precipitation for 1970 and 1971. For 1970, Mount Clemens recorded a rainfall near the average of the total annual precipitation for the Southeast Michigan Network. The year 1971 was a relatively "dry" year in southeastern Michigan with Mount Clemens reporting one of the lowest total annual precipitations in the region. The data for the Mount Clemens rain gauge station pertaining to hourly, daily, and monthly precipitation totals, as well as maximum precipitation for various time intervals, is consistent with data for the other rain gauge stations in the Southeast Michigan Network.

TABLE 1. SELECTED RAINFALL DATA FROM SOUTHEAST MICHIGAN NETWORK STATIONS (inches)

Year	Southeast Michigan average annual precip.	Mount Clemens annual precip.	Remarks
1970	26.16	25.93	54% of stations reported total annual precip. between 23.0 & 27.0 in.
1971	22.03	17.90	67% of stations reported total annual precip. between 20.0 & 24.0 in.

The post-construction study period lasted from April 15, 1972 through April 15, 1973. During this study period the Mount Clemens rain gauge station recorded 23.93 inches of precipitation. During this same period of time, the Shook Road Treatment Plant rain gauge station, located 3 miles south of the Mount Clemens rain gauge station, recorded 22.43 inches of total precipitation. Also, for this same period, the Metropolitan Beach main gauge station, located 4 miles southeast of the Mount Clemens rain gauge station, recorded 21.49 inches of precipitation. The differences between the rainfall characteristics of the Mount Clemens rain gauge station and the other rain gauge stations in the Southeast Michigan Network for the post-construction study period were not substantial.

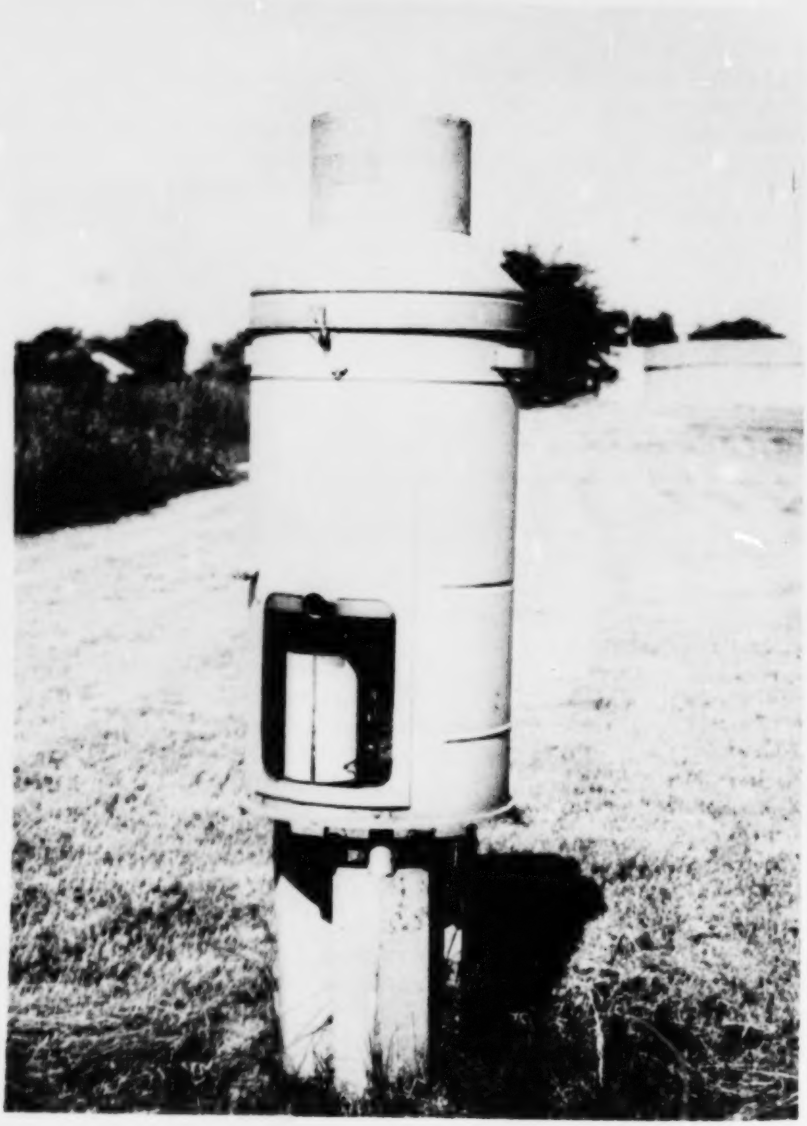


FIGURE 8. RAIN GAUGE STATION

SECTION IV

PRE-CONSTRUCTION STUDIES

OBJECTIVES

Pre-construction studies were initiated to indicate conditions which prevailed before the Demonstration Facility was placed into operation. The studies were performed from April 1, 1970 to September 30, 1971. Initially, these studies were to include the following:

- Monitoring combined sewer overflows for rate and volume at three existing overflow chambers, two of which were to be relieved by the construction of the Demonstration Facility.
- Collection and analysis of combined sewer overflow samples from these three locations.
- Collection and analysis of grab samples from the Clinton River at upstream and downstream (from project site) locations during any weather and storm conditions.
- Investigation of rainfall-runoff relationships -- only limited data were collected.

All of these objectives were substantially met except for the rainfall-runoff relationship investigations. Such investigations were impossible to accomplish due to the unusually dry year which occurred during this time period of the study. The average annual rainfall for the Clinton River Basin is approximately 30 inches while the actual rainfall experienced during the years of 1970 and 1971 were, again, 25.93 inches and 17.90 inches respectively. As a result of the low rainfall plus initial operational malfunctions, the quantity of rainfall and runoff data collected was not considered to be sufficient to allow for conclusive results.

DESIGN AND CONSTRUCTION

Three combined sewer overflow structures were modified to meet the objectives of the pre-construction studies. These structures were as follows:

Clemens Street Drain Overflow Structure.

Spruce Street Drain Overflow Structure.

Sewage Treatment Plant Influent Overflow Structure.

These monitoring locations were designated as sampling points 1, 2, 3 respectively as shown on Figure 7.

The modifications to the existing overflow structures included the replacement of the flap gate in the flap gate chamber with a check valve plus the installation of a magnetic flow meter for collecting rate and volume data. Figure 9 shows the layout of a typical overflow structure as modified for use in this project.

Magnetic Flow Meter and Check Valve

The 20 in. diameter magnetic flow meters were designed to be capable of operating submerged under a maximum head of 10 feet and to show a positive zero reading during conditions when they were not in use. The flow signal transmitting unit sent a 4-20 milliamp signal to be recorded in the sewage treatment plant. Figure 10 shows the Spruce Street Drain Diversion Chamber with the signal wire connection to the magnetic flow meter.

A 20 in. diameter horizontal swing check valve was used to prevent river water from flowing into the sewer system. Each structure was thus restricted from its original dimension to a 20 in. diameter discharge. This design provided the most accurate monitoring for intermediate flows.

Automatic Samplers (Serco)

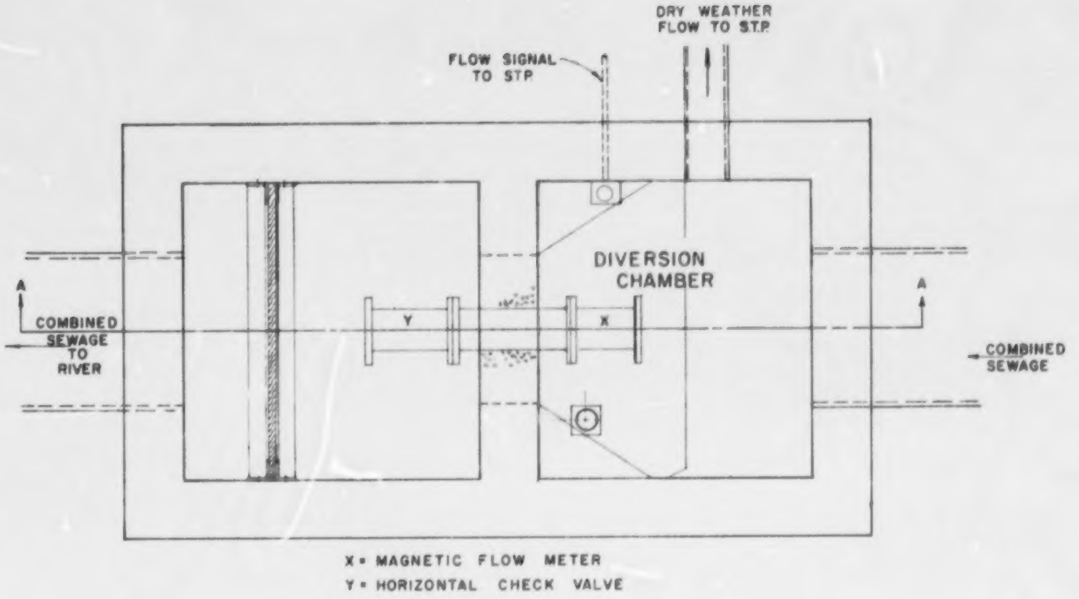
Each of the three overflow structures was equipped with an automatic sampler that was bolted to the top of the structures. The samplers were set up to collect 24 individual samples at 15 minute intervals during an overflow. The two samplers located at the Clemens Street Overflow Structure and the Spruce Street Overflow Structure were automatically activated by a 0.5 cfs flow through the magnetic flow meter. The third sampler, located at the sewage treatment plant influent overflow structure, was activated by a level control switch.

The samplers were designed to have the 24, 16-ounce sampler bottles evacuated by a vacuum pump to an evacuation of 27 in. of mercury. Sample tubes were extended to a point near the bottom of the diversion chambers such that, when an overflow was occurring, the evacuated bottles would draw a representative grab sample of combined sewer overflow upon the timed release of a pinched sample tube. Figure 10 shows the Spruce Street Drain Diversion Chamber with the sample tubes extending down along one side of the magnetic flow meter.

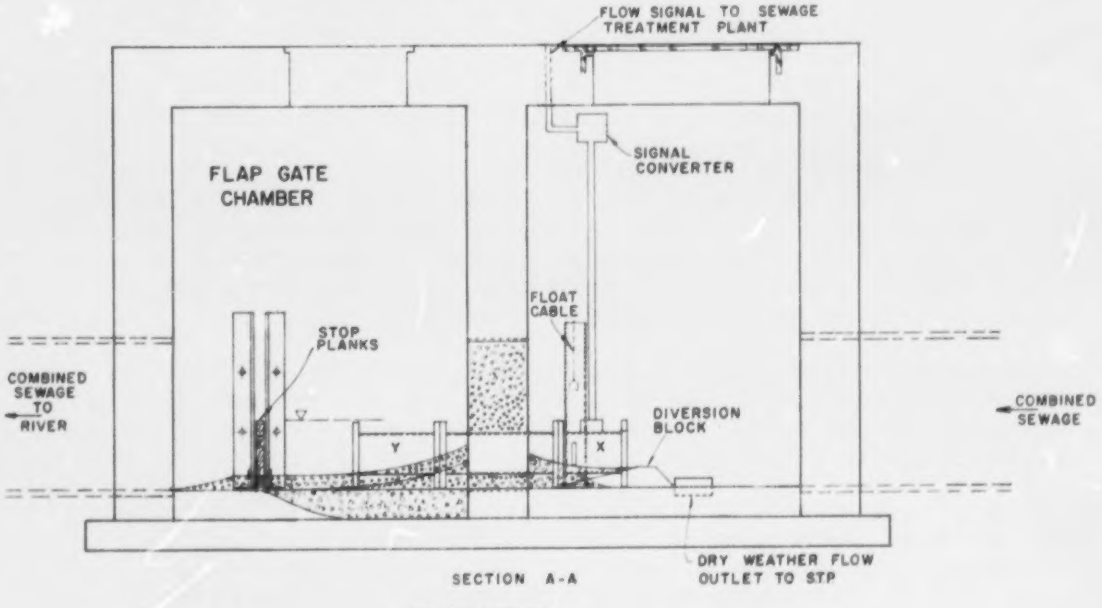
The automatic samplers stored the individual samples in a cooling compartment which kept the samples at a 4°C temperature. The refrigeration compartment and the control compartment of the Spruce Street overflow samplers can be seen in Figure 11.

Each sampler was secured to the top of its respective overflow chamber and was made as vandal proof as possible. The transmitting unit was fastened to the side of each sampler to make it readily accessible. Figure 12 shows the Clemens Street Drain sampling and transmitting unit.

The signals from the Clemens Street Drain Overflow Structure were transmitted over telephone wires while the signals from the other two structures were transmitted through buried wires since these structures were within the project site.


DRY WEATHER FLOW TO S.T.P.
FLOW SIGNAL TO STP
DIVERSION CHAMBER
A
A
COMBINED SEWAGE TO RIVER
Y
X
COMBINED SEWAGE
X = MAGNETIC FLOW METER
Y = HORIZONTAL CHECK VALVE

PLAN


FLOW SIGNAL TO SEWAGE TREATMENT PLANT
SIGNAL CONVERTER
FLAP GATE CHAMBER
FLOAT CABLE
STOP PLANKS
COMBINED SEWAGE TO RIVER
DIVERSION BLOCK
COMBINED SEWAGE
Y
X
SECTION A-A
DRY WEATHER FLOW OUTLET TO STP

ELEVATION

FIGURE 9. MODIFIED DIVERSION AND OVERFLOW STRUCTURE

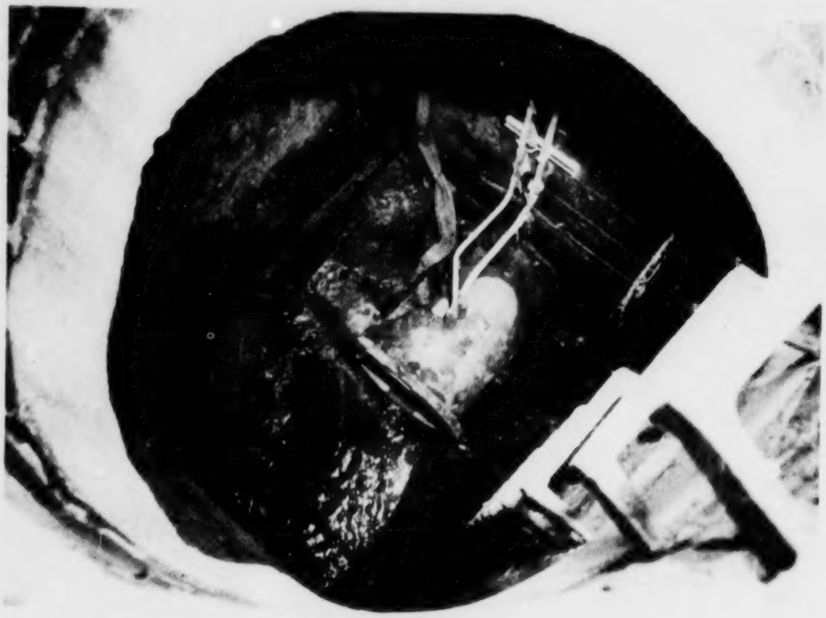


FIGURE 10. SPRUCE STREET DRAIN DIVERSION CHAMBER

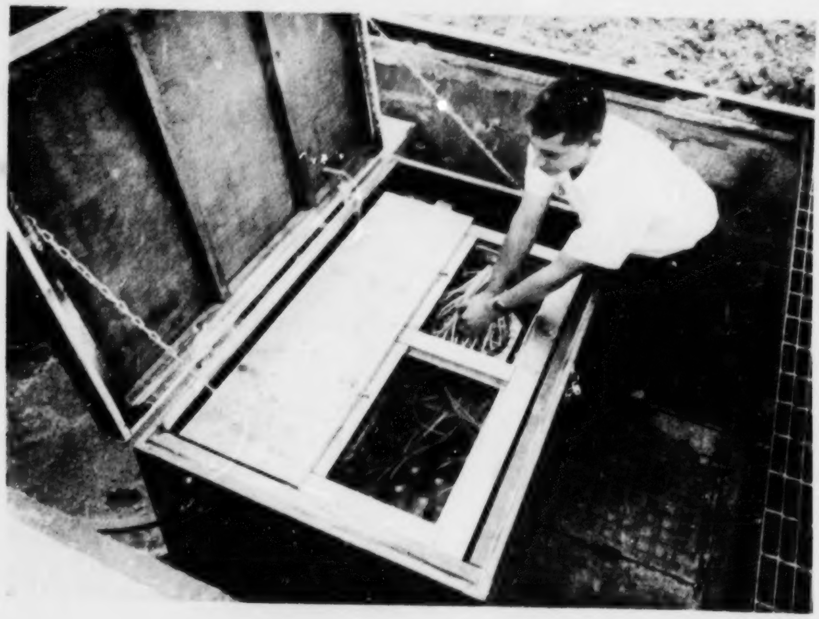


FIGURE 11. SPRUCE STREET DRAIN SAMPLER

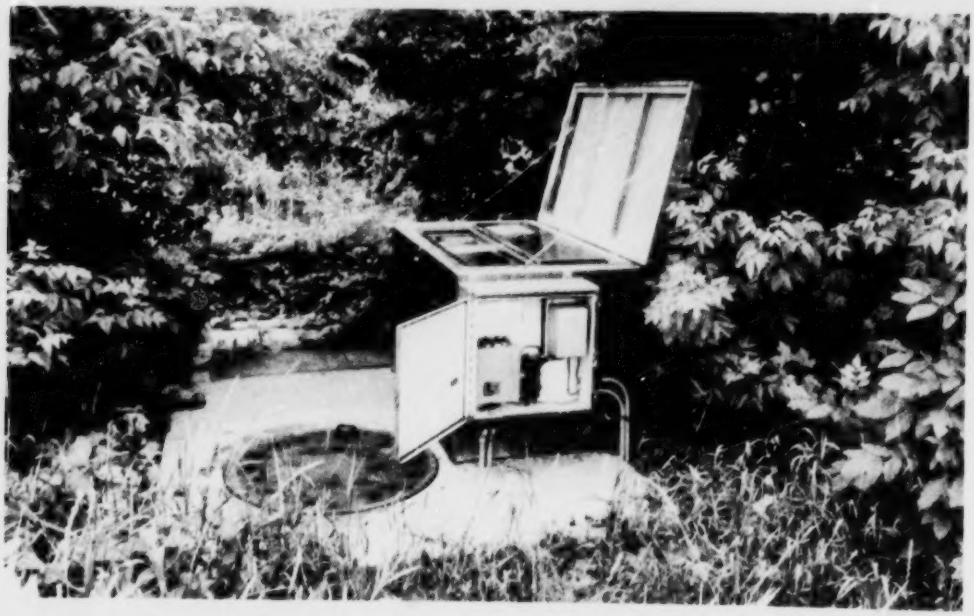


FIGURE 12. CLEMENS STREET DRAIN SAMPLER

OPERATION

The pre-construction sampling program was performed from April 1, 1970 to September 29, 1971, and the results are presented in Tables A-1 and A-2 of the appendix.

The start up of the Clemens Street Drain overflow monitoring station was initially a problem because of excessive resistance incurred in the telephone wires between the overflow structure and the recorder in the treatment plant. The excessive resistance prevented a sufficient signal from reaching the recorder. This problem was corrected after appropriate modifications were made to the transmitting unit.

Efficient automatic sampling of raw sewage has always been a problem predominantly due to inconsistencies of the wastewater. The sampling of combined sewer overflows presented a similar problem during the study for this project. Large particulate matter, pieces of cloth, and debris had the tendency to block the sample tubes during the suction cycle for a set of samples. This was the most common reason why several of the sets of samples presented in the appendix are incomplete.

The automatic samplers worked only during periods of overflow from the project service area when the rate exceeded 0.5 cfs. This eliminated a certain number of samples from being captured during low intensity storms. Moreover, during this portion of the project, one peculiarity of the Mount Clemens sewer system which affected the study was discovered. When the initial surcharge from storm water runoff was in the system and being conveyed from outside of the service area to the sewage treatment plant, some overflow structures would allow a discharge to go to the river before others would. These structures, which discharged overflows to the river, relieved the main plant influent line to the extent that a maximum surcharge from the two project service areas would flow to the plant instead of discharging to the river. It was determined that a rainfall intensity of 0.09 in./hr for 20 minutes was required to produce a 0.5 cfs overflow.

The existing overflow structure which was designed to relieve the sewage treatment plant influent line was found to exist at an elevation high enough such that it never functioned. The other overflow structures throughout the City were capable of completely relieving the main plant influent line during each overflow occurrence. Thus, no sampling data was ever obtained from this structure during pre- or post-construction studies.

Rainfall data were analyzed for the pre-construction period to determine the number of overflows which should have occurred. The analysis was based on the assumption that for an hour of rainfall, the entire service area for each sewer district would contribute to the overflow. Thus, the amount of rainfall in excess of 0.02 in./hr (the quantity which would surcharge to the treatment plant) for Clemens Street Drain would contribute to an overflow. Similarly, the amount of rainfall in excess of 0.05 in./hr (the quantity which would surcharge to the treatment plant) from Spruce Street would contribute to an overflow. For analysis purposes, it was arbitrarily assumed that a gap equal to or greater than 4 hours in the hourly rainfall records would constitute the initiation of a separate storm.

The analysis resulted in the "theoretical" number of overflows from the Clemens Street Drain amounting to 113, and for Spruce Street Drain the resulting number was 72. The number of overflows for the first 12 months of the study period "theoretically" amounted to 86 and 53 for the Clemens Street and Spruce Street Drains respectively.

The monitoring facilities recorded 33 overflow occurrences during the pre-construction period. For the first 12 months of the study 29 of the overflows were experienced, while during the remaining 6 months only 5 overflows were monitored.

The large discrepancy between the number of theoretical and monitored overflows probably resulted from two circumstances. First, the monitoring equipment occasionally malfunctioned which obviously produced some gaps in the data. Secondly, during short, low intensity storms, the runoff coefficient would be less than 0.5 because initially the ground is dry, and would absorb a great deal of the rainfall.

The number of overflows which were initially predicted to be monitored during a one year period was 40, or an average of 3.3 overflows per month. The 29 overflows monitored over the first 12 months averaged only 2.4 overflows per month. The 18 month average was only 1.9 overflows per month.

The total rainfall for the 18 month period was 34.30 inches which is less than the average 48.80 inches of rainfall for similar 18 month periods between 1969 and 1974.

31 overflow events were monitored at the Clemens Street Drain sampling point. Out of the 31 events, only 21 of them had suspended solids and 5-day BOD data and out of these 21, only 11 had rate/volume data.

24 overflow events were monitored at the Spruce Street Drain sampling location. Out of the 24 events, 14 had suspended solids and 5-day BOD data and out of these 14, only 7 had rate/volume data.

EVALUATION

Of the total number of actual storms causing overflows, no data of any sort were collected on approximately 20% of them due to operational failures of some sort. There were also instances when overflows did occur, but were of such small magnitude that the automatic sampler was not actuated and/or the recorder did not record a dependable reading. These factors along with the low rainfall during this period resulted in the deletion of the rainfall runoff investigations and the limited analysis of the overflow characteristics.

The combined sewer overflow rate and volume monitoring methods used during the pre-construction studies are not recommended for future projects. More sophisticated procedures are required for the accurate measuring of the widely varying rate and volume characteristics of combined sewer overflows.

> "...Conventional rate-of-flow meters have been developed mainly for relatively steady-state irrigational, stream, and sanitary flows and not for the highly varying surges encountered in storm and combined sewers..." [5]

The concept of monitoring combined sewer overflows by capturing a series of grab samples throughout each overflow event is strongly recommended. This type of sampling system will indicate the drastic changes in composition which the wastewater experiences during typical overflow events. However, investigations should be performed to develop a failsafe method of obtaining these samples without the interference of debris and other foreign material in the wastewater.

Of the 33 overflows that were monitored during the pre-construction studies, 14 of them are summarized in Table 2. These 14 overflows were selected as having the most complete data. The overflow data presented in these two tables includes the total quantity of reliable rate and volume data collected.

TABLE 2. COMBINED SEWER OVERFLOW CHARACTERISTICS

Date of overflow, mo/day/yr	Total precip., inches	Duration of storm, hours	Service district	Theoretical volume of overflow, cu. ft.	Recorded volume of overflow, cu. ft.	Recorded duration of overflow, hours	Recorded peak flow rate, cfs	Initial SS, mg/1	Average SS, mg/1	No. of Samples	Initial BOD_5 mg/1	Average BOD_5 mg/1	No. of samples
12/19/70	(melting snow)		Spruce	-	21,248	4.2	2.5	336	382	10	124	101	10
1/3-4/71	0.27	9	Clemens Spruce	32,822 3,267	177,065 19,730	9.1 5.0	9.4 1.5	736 584	616 420	20 19	258 264	190 155	20 19
2/4-5/71	1.32	12	Clemens Spruce	331,208 147,015	215,513 184,142	15.0 22.0	9.1 2.2	44 328	202 294	8 14	37 139	44 216	8 14
2/17/71	0.21	8	Clemens Spruce	29,838 1,634	103,204 24,283	6.1 4.2	8.1 1.6	568 308	334 394	18 22	171 100	102 131	18 22
2/18-19/71	0.18	6	Clemens Spruce	47,742 6,534	115,350 50,084	7.0 8.0	5.7 9.6	276 212	233 246	23 20	110 150	81 115	23 20
2/19-20/71	0.34	8	Clemens Spruce	56,693 6,534	511,982 237,772	16.0 16.0	9.0 1.8	- -	- -		- -	- -	
2/22/71	0.50	10	Clemens Spruce	113,387 40,838	67,793 24,283	7.1 8.7	9.6 0.9	368 340	516 548	19 17	59 32	46 52	19 17
3/6-7/71	0.52	16	Clemens Spruce	74,597 4,901	98,145 14,165	13.4 9.3	8.2 0.7	300 160	289 288	20 19	74 53	91 64	20 19
3/13/71	(melting snow)		Clemens Spruce	- -	13,356 21,046	4.0 3.0	5.3 2.0	424 -	286 -	12	117 -	89 -	12
3/19/71	0.15	9	Clemens	11,935	14,166	4.7	5.2	396	387	11	206	140	11
5/19/71	0.35	4	Clemens Spruce	80,564 27,770	24,284 34,402	3.8 3.0	4.5 3.5	232 -	287 -	10	140 -	128 -	10
5/24/71	0.25	17	Clemens	32,822	5,059	2.0	5.1	568	430	2	276	231	2
6/13/71	0.26	4	Clemens	65,645	23,272	2.5	9.0	644	344	14	249	179	14

This table shows the duration of the rainfall and the accumulated precipitation (in inches of rainfall) for each particular rainfall which caused an overflow. The rainfall data were obtained from the rain gaging station at the wastewater treatment plant. The duration, volume, and peak flow rate for the overflows were determined from the strip charts as recorded from the magnetic flow meters. The suspended solids and BOD_5 values were determined from analyses done on the time sequenced grab samples collected during the course of each overflow.

Comparison of the recorded volumes versus the theoretical volumes of overflow did not show good correlation. Several circumstances could have contributed to the inconsistent results. Note first, that each time the recorded overflow volume exceeded the theoretical overflow volume, the overflow event took place during a winter month. Hence the imperviousness was most likely more than 0.6 and the incidence of snow melting probably also contributed greatly.

The calculation for the theoretical volume of overflow was based on a total accumulation which did not take into account the effect of surcharge during long, low intensity storms. For example on 5/24/71, rainfall accumulated to 0.25 inches for a 17 hour duration storm. For four non-consecutive one-hour periods, the intensity was 0.01 in./hr, one hour had a 0.02 in./hr intensity, while other one hour intensities were 0.03, 0.04, 0.05, and 0.07. The lower intensity periods would thus surcharge to the treatment plant and not cause an overflow.

The runoff coefficient of 0.6, which was assumed to be the average value for total runoff, seems to be questionable. The fact that this coefficient varies with respect to different circumstances (such as frozen ground and longer duration storms) makes the direct calculations of runoff volumes, as performed for this report, not realistic. An in depth investigation would be required to ascertain an accurate time function runoff coefficient for this area. Investigations have been performed for similar areas in other parts of the country to develop accurate runoff coefficients. Also, the limited data which is available from this study is not sufficient for conclusive results. Further calculations and analysis in this area was therefore considered to be beyond the scope of this report.

It was considered to be most beneficial to present a general overview of the available sampling data from the two sewer districts. Thus, the concentrations of BOD_5 and SS were plotted for each overflow point using the most complete sets of sampling data available. These graphs are shown in Figures 13 through 16. Table 2 identifies the plotted overflow events for each figure.

The suspended solids concentrations from each overflow structure generally varied between 75 mg/l and 900 mg/l. Also, each structure showed peaks approaching 1200 mg/l.

The BOD_5 concentrations from each overflow structure generally varied between 50 mg/l and 250 mg/l with peaks occurring at a concentration of approximately 300 mg/l.

TABLE 3. OVERFLOW EVENT IDENTIFICATIONS FOR FIGURES 13 THROUGH 16

Date of overflow, mo/day/yr	Accumulated rainfall, inches	Overflow event identification numbers			
		Fig. 13	Fig. 14	Fig. 15	Fig. 16
4/2/70	1.44		1		1
6/11/70	0.65	1		1	
6/18/70	0.49	2		2	
6/24/70	0.35		2		2
12/19/70	(snow melt)		3		3
2/4/71	1.60		4		4
2/17/71	0.18	3	5	3	5
2/18/71	0.05	4	6	4	6
2/22/71	0.55	5	7	5	7
3/6/71	0.74	6	8	6	8
3/13/71	(snow melt)	7		7	
3/19/71	0.27	8		8	

By observation, the graphs show no apparent regularities (other than general ranges of concentration) from which conclusive statements could be derived. A sophisticated approach incorporating an abundance of data would thus be required for the thorough analysis of combined sewer overflow characteristics.

An estimated annual loadings contributed by the service area to the receiving stream was calculated. The arithmetic mean of the total quantity of available data was used in this calculation. The average concentration of BOD_5 was thus found to be 140 mg/1 and for SS the concentration was determined to be 350 mg/1.

The annual runoff from the service area, assuming an average annual rainfall of 30 inches, would be as follows:

(30 inches/year)(0.6 runoff coefficient)(212 acres)(43560 sq ft/acre)/(12 inches/foot) = 13,852,080 cu ft/year or 3.6 cu ft/acre inch runoff.

The annual loading of SS from the service area would be:

(350 mg/1)(62.4 lbs/ft^3)(13,852,080 cu ft/year)/10^6 = 302,529 lbs/year,

or 1427 lbs/acre/year or 79 lbs/acre inch runoff.

The annual loading of BOD_5 from the service area would be:

(140 mg/l)(62.4 lbs/ft^3)(13,852,080 cu ft/year)/10^6 = 121,011 lbs/year

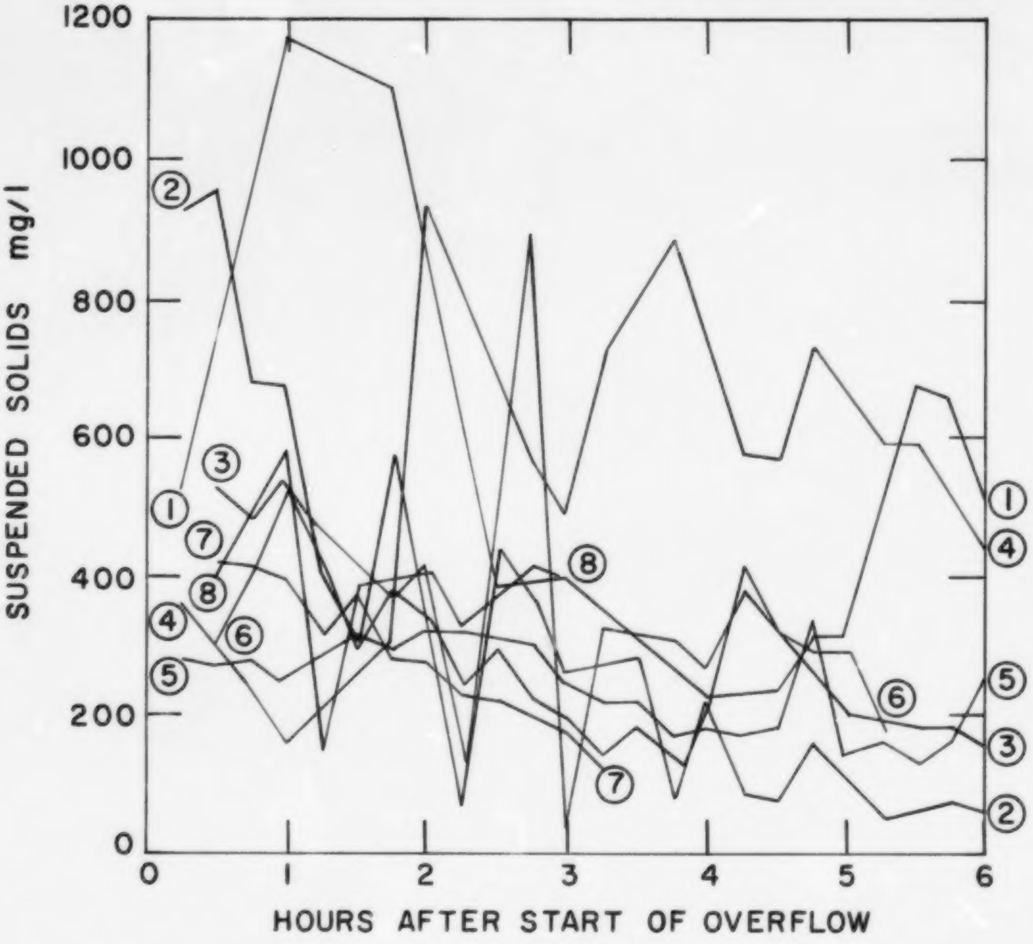
or 570 lbs/acre/year or 32 lbs/acre inch runoff.

Several comprehensive studies have been performed by others to determine rainfall-runoff relationships regarding combined sewer overflows.[6,7]

The BOD_5 and SS loadings from the Mount Clemens demonstration facility service area compare favorably with those in the referenced reports.

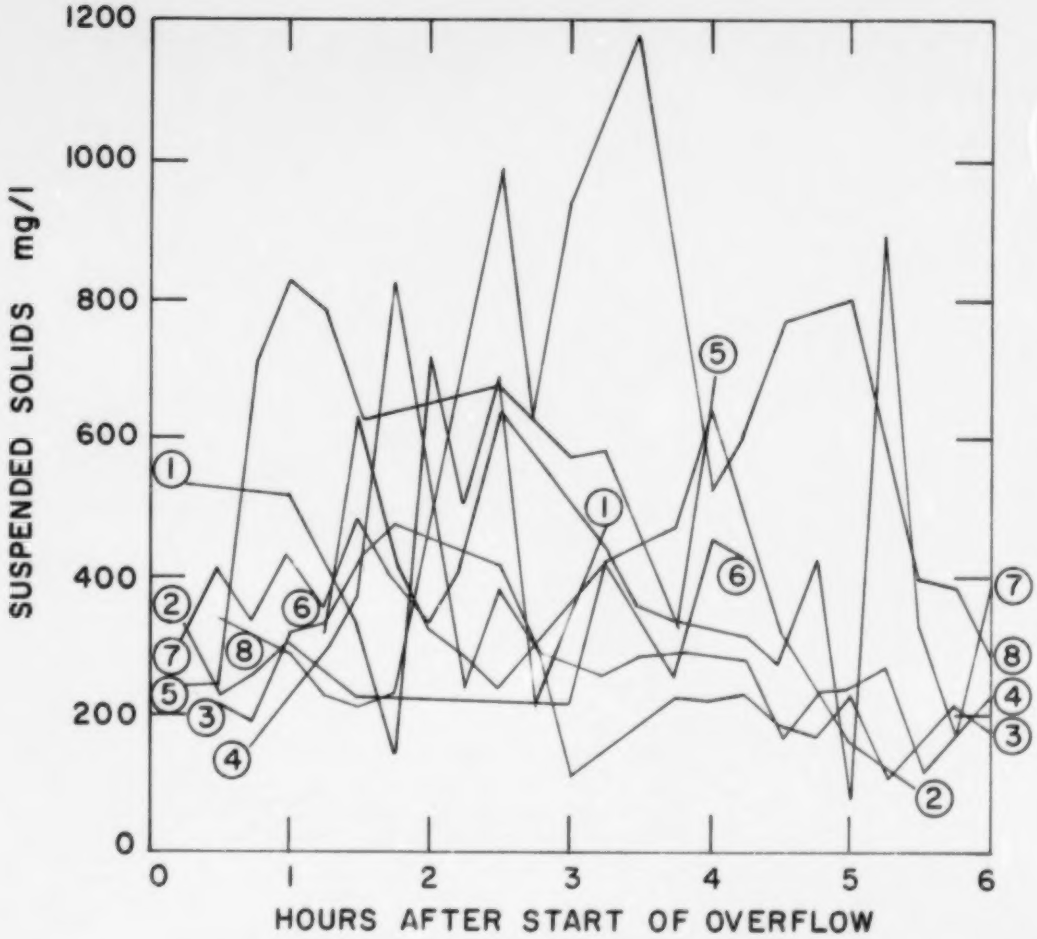
River sampling data were collected throughout the pre-construction studies. This data is presented in Appendix C, Table C-1. This data was used along with the data obtained during the post-construction studies to evaluate the effect of overflows on river quality. The evaluation is discussed in Section VI of this report.

FIGURE 13. CLEMENS STREET DRAIN



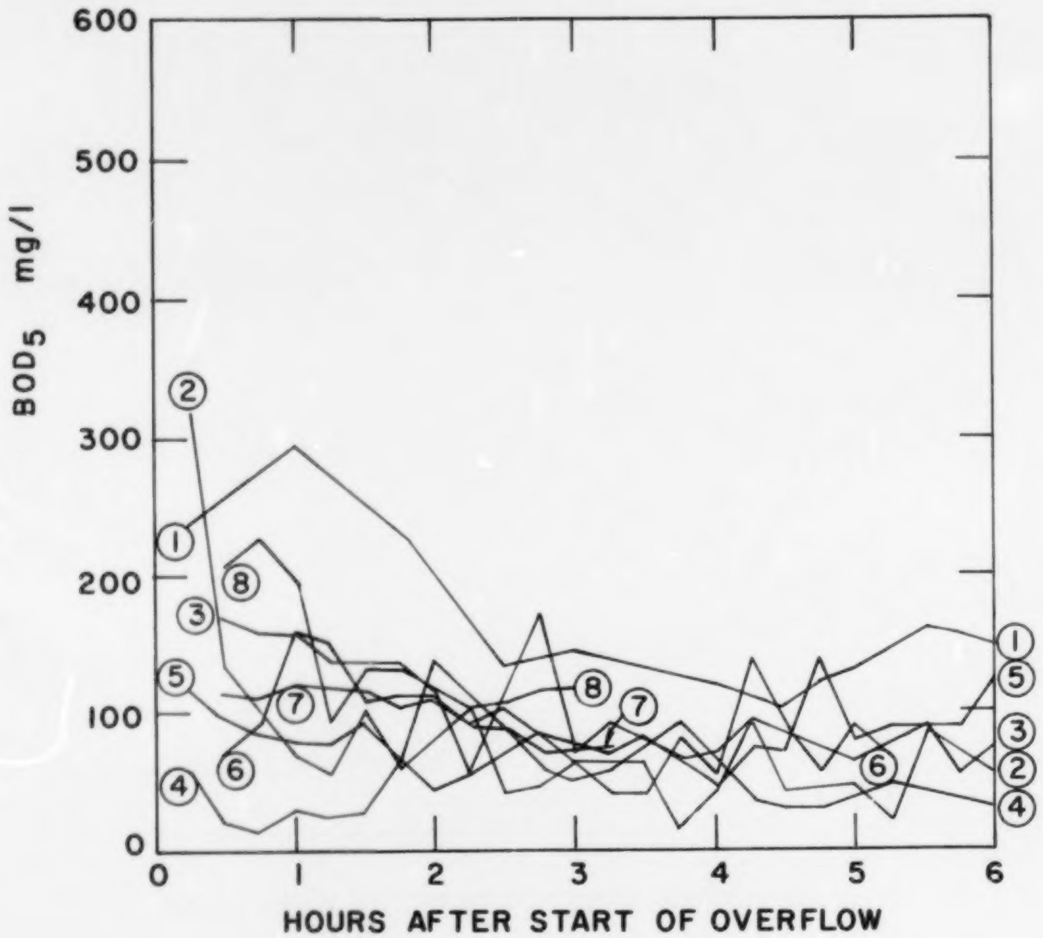
SUSPENDED SOLIDS CONCENTRATION
VS
TIME

FIGURE 14. SPRUCE STREET DRAIN



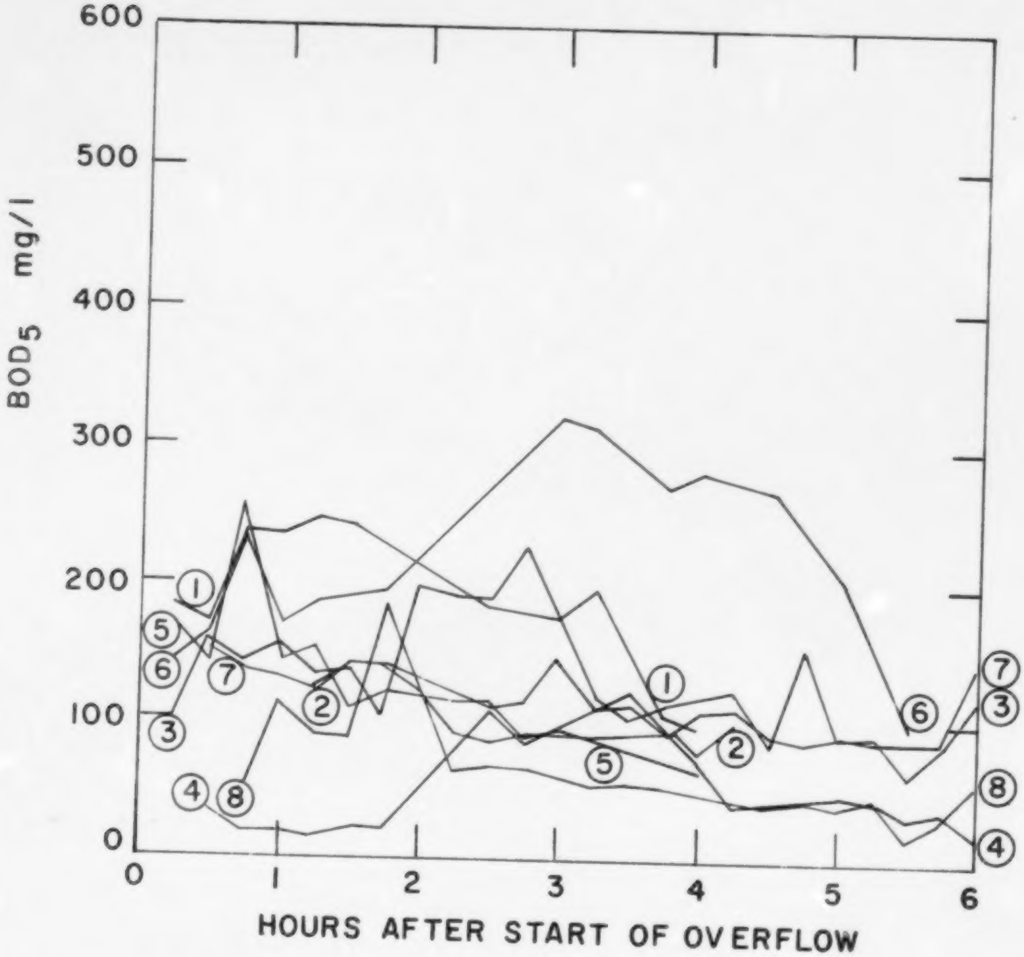
SUSPENDED SOLIDS CONCENTRATION
VS
TIME

FIGURE 15. CLEMENS STREET DRAIN



BOD$_5$ CONCENTRATION
VS
TIME

FIGURE 16. SPRUCE STREET DRAIN



BOD_5 CONCENTRATION
VS
TIME

SECTION V

FACILITY DESIGN AND CONSTRUCTION

CONCEPT

The design of the demonstration facility was based on the concept of a Multi-purpose Combined Sewer Overflow Treatment Facility. This basic concept materialized into a treatment facility that would be intermixed with an open reservation of park land area having ponds, or lakelets, containing a renovated water of sufficient quality to be acceptable for limited recreational purposes. Moreover, it was contemplated that some of the renovated process water could be used to satisfy the moisture demands of the park greenery while incidentally satisfying part of its nutrient demand.

There are several factors which tend toward making a combined sewer overflow wastewater more susceptive for use in this concept as compared to using a dry-weather-flow wastewater which are as follows:

> In this particular area's climate, rainstorms occur intermittently during the summer, and they occur very infrequently during the winter. This factor allows some of the treatment units to be completely retired during the winter months. Moreover, it could allow some of the treatment units to be placed in an outside unsheltered area without the need for protection from freezing. On the other hand, regular sewage (or dry-weather-flow) occurs inexorably both winter and summer and becomes much more difficult to handle during the winter months because of the continuous flow and continued bio-chemical oxygen demand under the ice cover of the lakes in the winter time.
>
> Upon removal of the settleable material from a combined sewer overflow, especially during the earlier part of any storm, the storm water can be readily partially renovated by aeration during its extended containment period in a retention basin, prior to its release to a lake system.
>
> Rain water, properly controlled, is a valuable resource which should be conserved and used as an urban asset rather than a liability.

The treatment process design is illustrated in the flow diagram shown on Figure 17. This figure also indicates the sampling locations which were used during the post-construction studies.

The treatment facility was designed to provide treatment to the overflows by means of a series of aerated lakelets with intermediate microstraining, disinfection, and high-rate pressure filtration prior to discharge into the Clinton River.

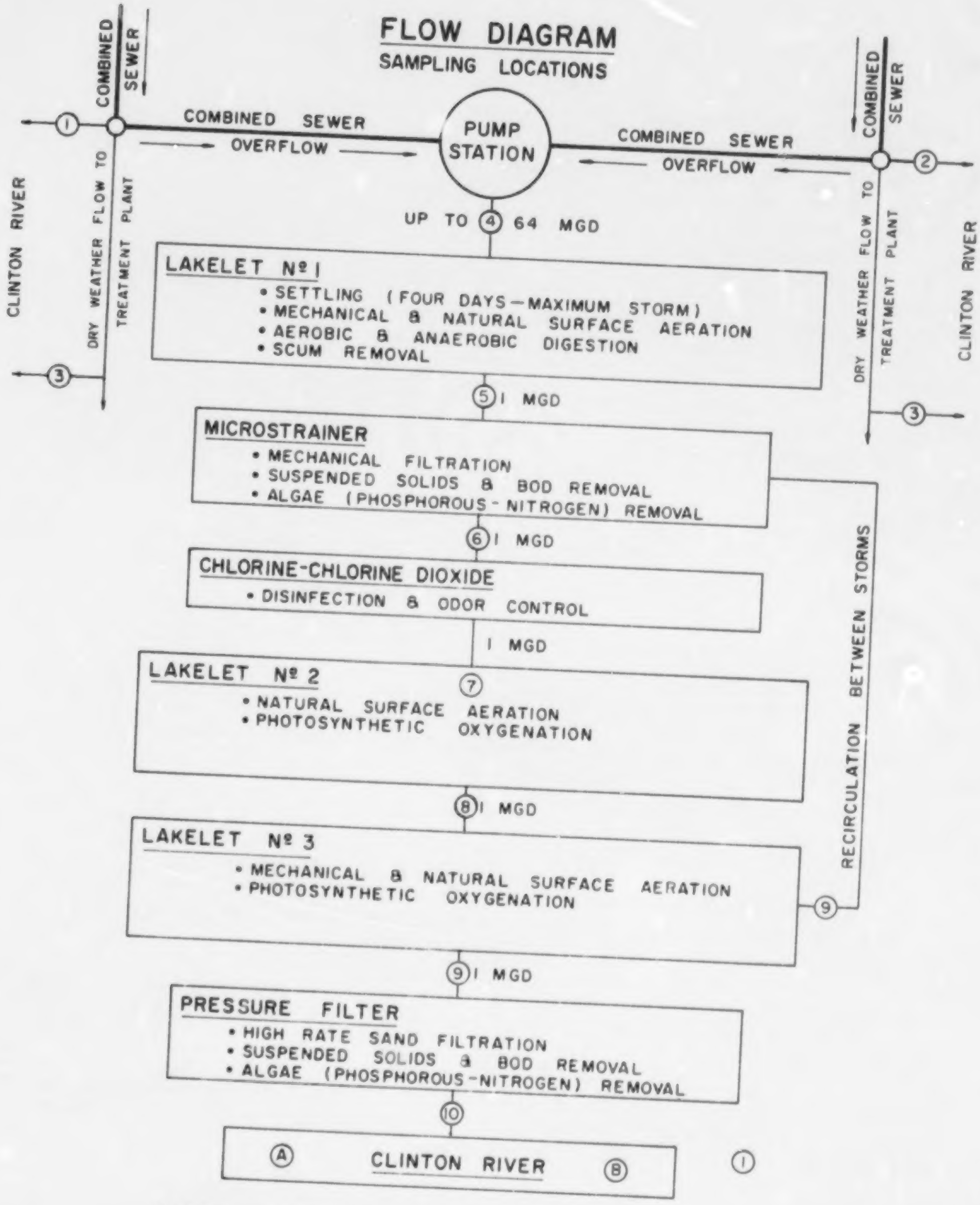

FLOW DIAGRAM
SAMPLING LOCATIONS
COMBINED SEWER
COMBINED SEWER
OVERFLOW
PUMP STATION
COMBINED SEWER
OVERFLOW
COMBINED SEWER
①
②
CLINTON RIVER
DRY WEATHER FLOW TO TREATMENT PLANT
③
DRY WEATHER FLOW TO TREATMENT PLANT
CLINTON RIVER
③
UP TO ④ 64 MGD
LAKELET Nº 1
• SETTLING (FOUR DAYS—MAXIMUM STORM)
• MECHANICAL & NATURAL SURFACE AERATION
• AEROBIC & ANAEROBIC DIGESTION
• SCUM REMOVAL
⑤ 1 MGD
MICROSTRAINER
• MECHANICAL FILTRATION
• SUSPENDED SOLIDS & BOD REMOVAL
• ALGAE (PHOSPHOROUS-NITROGEN) REMOVAL
⑥ 1 MGD
CHLORINE-CHLORINE DIOXIDE
• DISINFECTION & ODOR CONTROL
1 MGD
⑦
LAKELET Nº 2
• NATURAL SURFACE AERATION
• PHOTOSYNTHETIC OXYGENATION
⑧ 1 MGD
LAKELET Nº 3
• MECHANICAL & NATURAL SURFACE AERATION
• PHOTOSYNTHETIC OXYGENATION
RECIRCULATION BETWEEN STORMS
⑨
⑨ 1 MGD
PRESSURE FILTER
• HIGH RATE SAND FILTRATION
• SUSPENDED SOLIDS & BOD REMOVAL
• ALGAE (PHOSPHOROUS-NITROGEN) REMOVAL
⑩
Ⓐ
CLINTON RIVER
Ⓑ
①

FIGURE 17. DEMONSTRATION FACILITY FLOW DIAGRAM

The area adjacent to the wastewater treatment plant was chosen as the project site for the following reasons:

the area was easily accessible by the treatment plant staff.

two combined sewer districts could be intercepted within this project site.

the area was an unsightly dump which needed to be cleaned up and restored to usefulness.

The treatment facility site plan is shown on Figure 18. By studying this site plan with the help of the flow diagram (Figure 17) the process description can be followed.

TREATMENT PROCESS DESCRIPTION AND DESIGN

The design of major process units was based on the design parameters presented in Table 4. The following process unit descriptions show how each unit was designed for application in the Mount Clemens concept.

Overflow Chambers

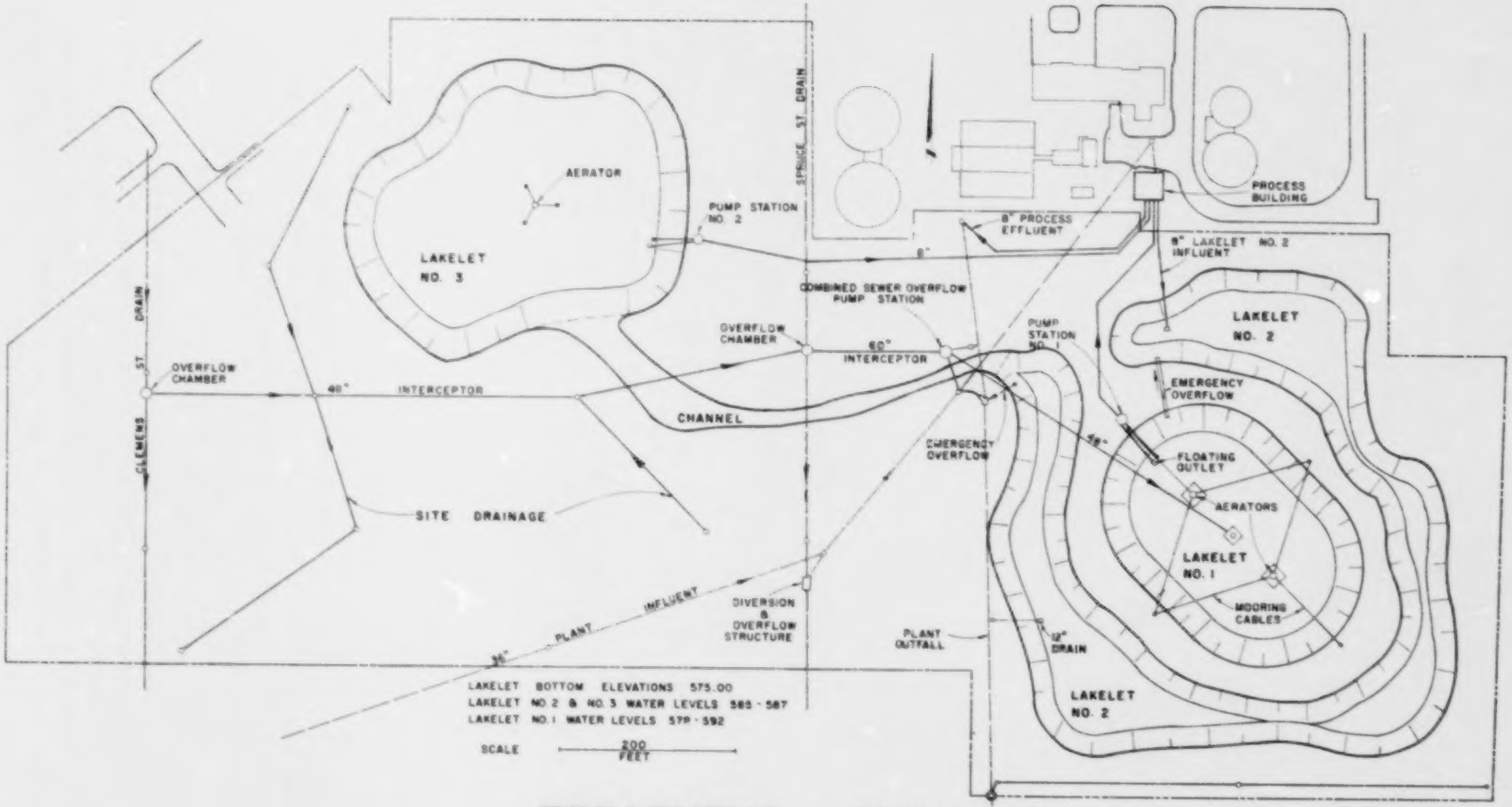
The overflow chambers on the Clemens Street and the Spruce Street Drains were designed to intercept the combined sewer overflows while allowing dry-weather flows to flow through for treatment at the existing trickling filter sewage treatment plant.

Figure 19 shows the overflow structure as designed for the Spruce Street Drain. (This particular structure included the pump screen which prevented large solids from damaging the storm water pumps in the combined sewer overflow pump station.) The troughs for each structure were 12 inches wide. Each would carry a surcharge of approximately 3 cfs to the treatment plant before and during the course of an overflow. Primarily however, the troughs were designed to convey the dry-weather flow plus a minimum surcharge to the treatment plant.

The overflow structures were designed to intercept the maximum capacity of the combined sewers. Thus, the only circumstance which would create an overflow to the river would be the result of the interceptor's refusal to accept flows from the overflow structure. In such an event, the excess overflows would be discharged to the river and be monitored by the magnetic flow meters and automatic samplers which were already in place to establish baseline data from the pre-construction period.

Interceptors

Interceptors conveyed overflows to the combined sewer overflow pump station for pumping into Lakelet No. 1. The maximum anticipated rate of flow was determined, for design purposes, by the rational method, as indicated in Table 4. It was determined that the Mount Clemens combined sewers were originally designed for approximately a two year storm. A two year storm would result in a rainfall intensity (I) equal to 1 in./hr at 55 minutes


LAKELET NO. 3
AERATOR
PUMP STATION NO. 2
SPRUCE ST DRAIN
8" PROCESS EFFLUENT
8"
PROCESS BUILDING
8" LAKELET NO. 2 INFLUENT
COMBINED SEWER OVERFLOW PUMP STATION
OVERFLOW CHAMBER
60"
INTERCEPTOR
PUMP STATION NO. 1
LAKELET NO. 2
EMERGENCY OVERFLOW
CLEMENS ST DRAIN
OVERFLOW CHAMBER
48"
INTERCEPTOR
CHANNEL
EMERGENCY OVERFLOW
48"
FLOATING OUTLET
AERATORS
LAKELET NO. 1
MOORING CABLES
SITE DRAINAGE
DIVERSION & OVERFLOW STRUCTURE
36" PLANT INFLUENT
PLANT OUTFALL
12" DRAIN
LAKELET NO. 2
LAKELET BOTTOM ELEVATIONS 575.00
LAKELET NO. 2 & NO. 3 WATER LEVELS 585 - 587
LAKELET NO. 1 WATER LEVELS 579 - 592
SCALE
200 FEET

TREATMENT FACILITY
SITE PLAN

FIGURE 18

TABLE 4. PROCESS DESIGN PARAMETERS

Item no.	Process unit	Design parameter	Remarks
1	Clemens Street Drain	Existing-(29"x52" box sewer, 60 cfs).	Serves 137 acres within the city.
2	Spruce Street Drain	Existing-(42" sewer, 40 cfs).	Serves 75 acres within the city.
3	Combined sewer overflow interceptor	Rational method, Q=CIA.	I=1 in./hr @ 55 min C=0.5, runoff coeff.
4	Combined sewer overflow pump station	Rational method, Q=CIA.	3 pumps - 2 @ 18,000 gpm, 1 @ 9,000 gpm; 100 cfs peak.
5	Lakelet No. 1	1.5" rainfall accumulation, runoff coefficient = 0.6, 1 to 4 days detention.	1.5 acres, 0.75 million cubic feet.
6	Aerators	3.1 lbs O_2/hr.	(2)-20 hp floating aerators.
7	Outlet	1 mgd floating outlet.	Submerged intake leaves scum in Lakelet No. 1.
8	Pump station no. 1	1 mgd.	Controlled by Lakelet No. 1 water level.
9	Microstrainer	10 gpm/ft^2; 1 mgd.	6 ft diameter drum.
10	Chlorination	10 mg/l maximum dosage.	0-100 ppd chlorinator supplied.
11	Chlorine dioxide	Generation from sodium chlorite.	Sufficient data for comparison was not collected on ClO_2.
12	Lakelet No. 2	8 days detention, 10 feet deep.	2.8 acres, 1.1 million cubic feet.
13	Lakelet No. 3	7 days detention, 9 feet deep.	2.3 acres, 0.93 million cubic feet.
14	Pump station no. 2	1 mgd.	Lowhead pump for recirculation through microstrainer, high head for sand filter.

TABLE 4 (continued). PROCESS DESIGN PARAMETERS

Item no.	Process unit	Design parameter	Remarks
15	Sand filter	15 gpm/ft^2	2-stage pressure filter, 2-stage backwash with filtered water.

time of concentration. For the project area of 212 acres under study, the runoff coefficient was assumed to be 0.5. Thus the maximum anticipated rate of flow, or runoff, was determined to be 106 cfs.

In checking the capacities of the two sewers which were to be intercepted it was found that the Clemens Street Drain was 29" x 51" and had a maximum capacity of 60 cfs, the Spruce Street Drain was 42" in diameter and had a maximum capacity of 40 cfs.

Not all of the combined sewage would overflow and consequently the rate of runoff minus the surcharged flow continuing on into the sanitary sewer system gave the maximum rate of overflow. The maximum rate of overflow was determined to be approximately 100 cfs or 45,000 gpm.

Combined Sewer Overflow Pump Station

The combined sewer overflow pump station was designed to handle the maximum rate of overflow (100 cfs or 45,000 gpm) as described above.

The pump station pumping capacity increased in increments of 9,000 gpm (20 cfs). In order to do this, the station was equipped with three pumps as shown in Figure 20. Pump number one had a 9,000 gpm capacity while pumps two and three each had an 18,000 gpm capacity. The pumps were controlled by electrodes sensing the rising level in the wet well.

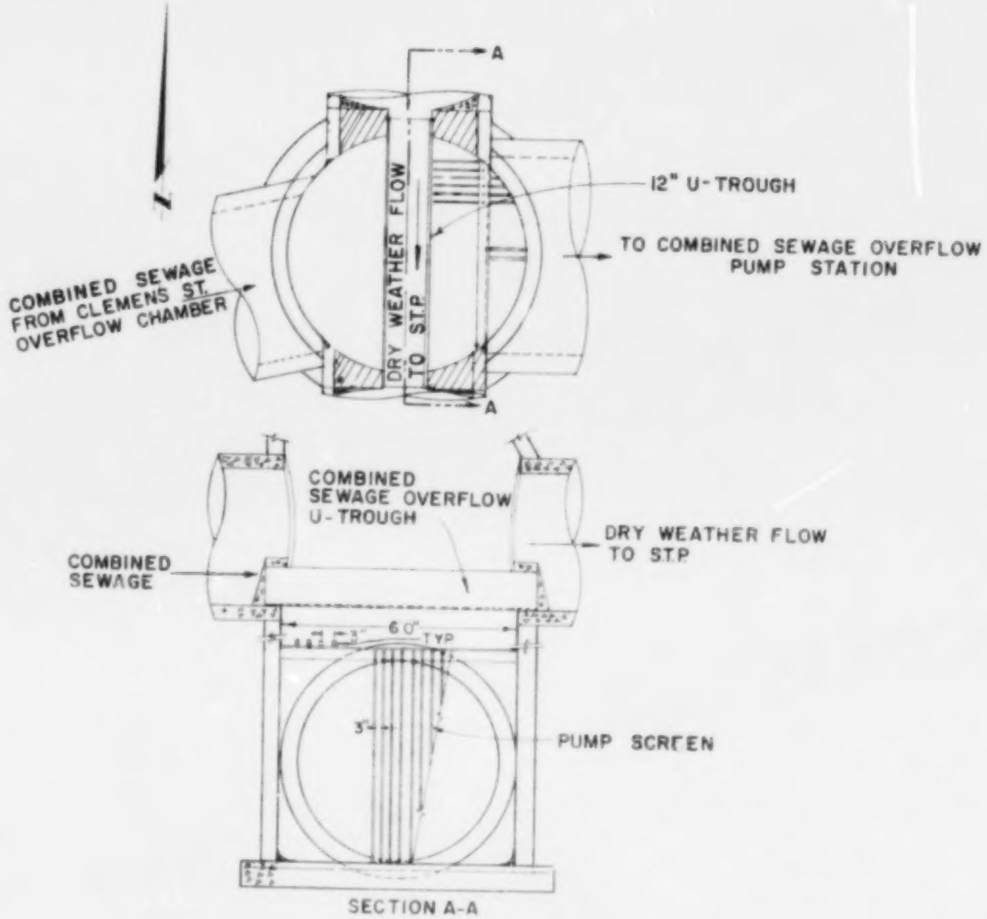
The electrodes were set at such an elevation as to allow the use of a portion of the 60" interceptor as additional wet well volume. Figure 21 shows a sectional view of the pump station including connections from the wastewater treatment plant effluent line plus the wastewater treatment plant influent line. These connections were included for operational flexibility. Figure 22 shows the installation of the propeller type pump.

The combined sewer overflows discharged from the pump station into a 48" pipe that siphoned the overflows into Lakelet No. 1.

Automatic Flow Proportioned Sampler (N-Con Systems)

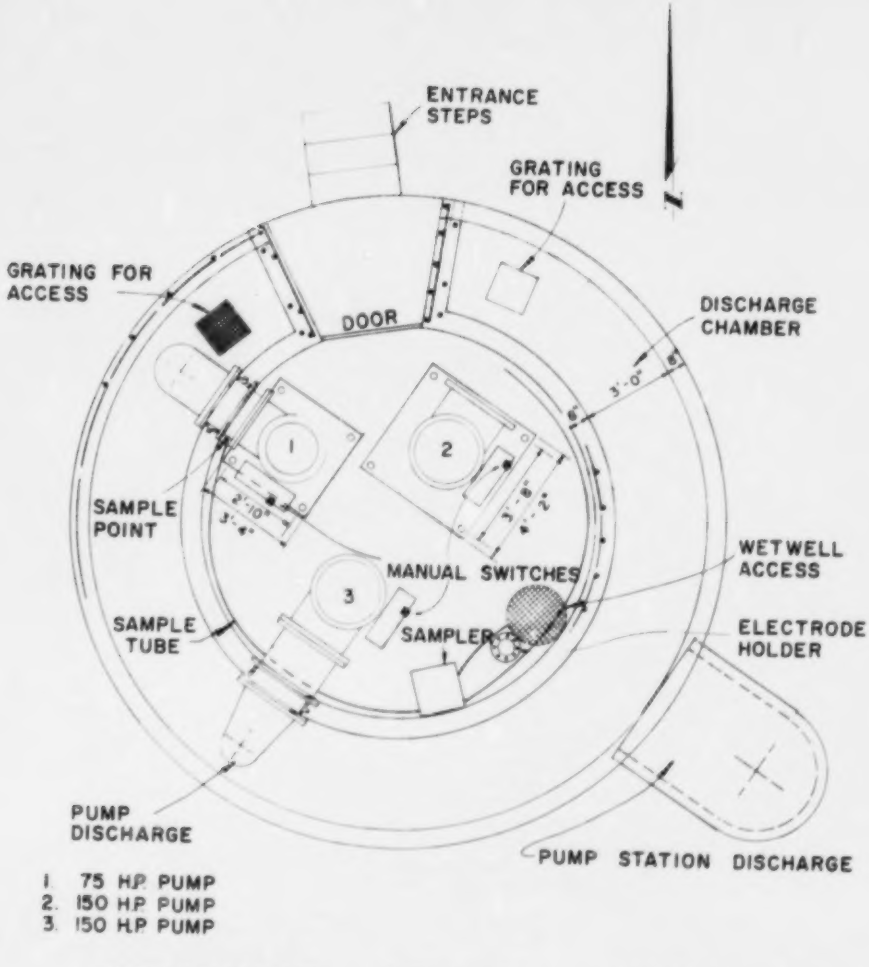
The combined sewer overflow pump station included a sampling point (No. 4) as shown on Figure 17. An automatic flow proportioned composite sampler was used to obtain baseline data for the treatment evaluation portion of this report.

The sampler received flows from the pump discharge pipes for the time the pumps were in operation. A 1 1/2" PVC pipe conveyed the combined sewer


A
12" U-TROUGH
DRY WEATHER FLOW TO S.T.P
TO COMBINED SEWAGE OVERFLOW PUMP STATION
COMBINED SEWAGE FROM CLEMENS ST. OVERFLOW CHAMBER
A
COMBINED SEWAGE OVERFLOW U-TROUGH
DRY WEATHER FLOW TO S.T.P
COMBINED SEWAGE
6 0"
TYP
3"
PUMP SCREEN
SECTION A-A

SPRUCE STREET DRAIN
OVERFLOW CHAMBER

FIGURE 19


ENTRANCE STEPS
GRATING FOR ACCESS
GRATING FOR ACCESS
DOOR
DISCHARGE CHAMBER
3'-0"
6"
6"
1
2
3
2'-10"
3'-4"
3'-8"
4'-2"
SAMPLE POINT
MANUAL SWITCHES
WETWELL ACCESS
SAMPLE TUBE
SAMPLER
ELECTRODE HOLDER
PUMP DISCHARGE
PUMP STATION DISCHARGE
1. 75 H.P. PUMP
2. 150 H.P. PUMP
3. 150 H.P. PUMP

COMBINED SEWER OVERFLOW PUMP STATION PLAN

FIGURE 20

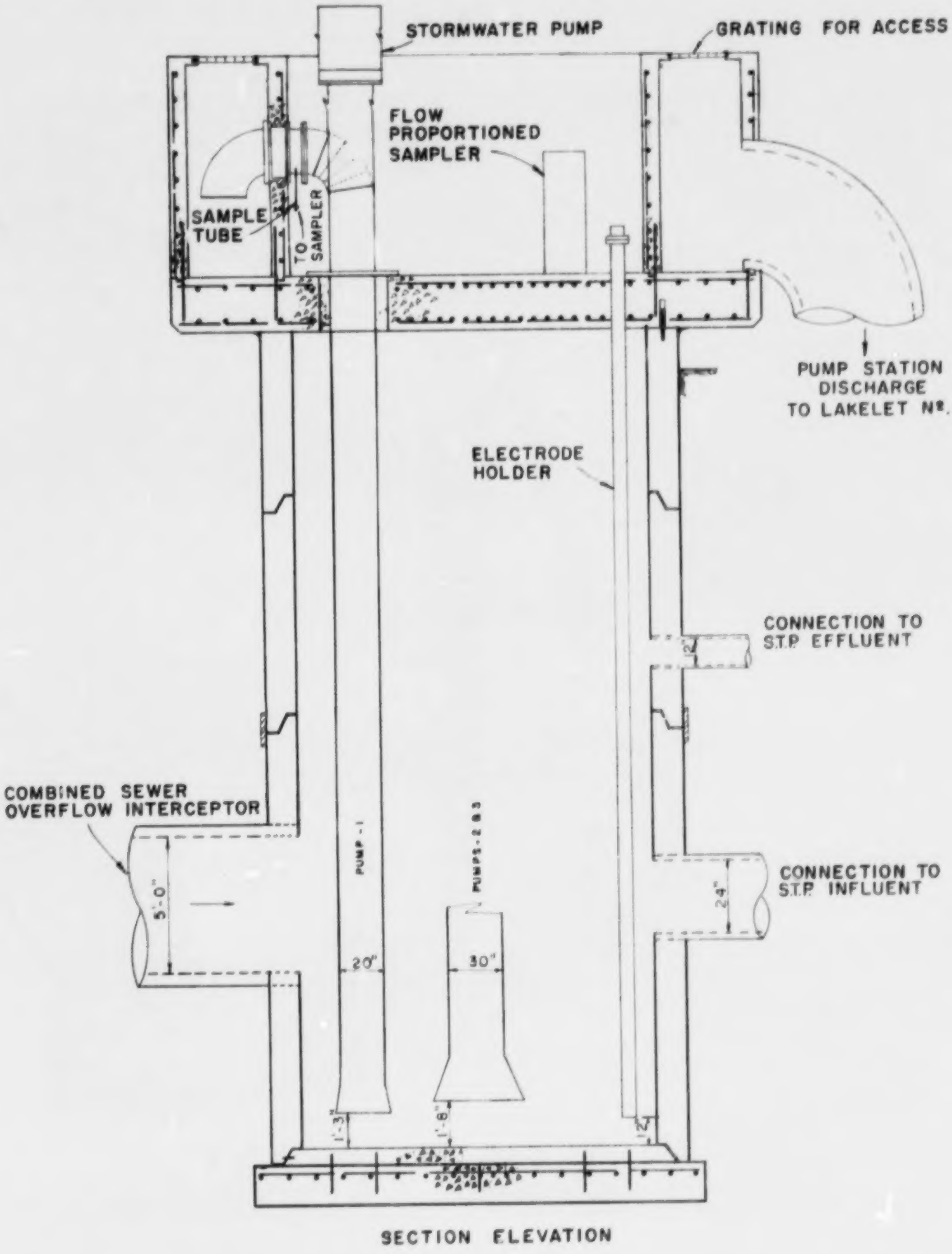

STORMWATER PUMP
GRATING FOR ACCESS
FLOW PROPORTIONED SAMPLER
SAMPLE TUBE
TO SAMPLER
PUMP STATION DISCHARGE TO LAKELET Nº.1
ELECTRODE HOLDER
CONNECTION TO S.T.P. EFFLUENT
12"
COMBINED SEWER OVERFLOW INTERCEPTOR
5'-0"
PUMP-1
PUMPS-2 & 3
20"
30"
CONNECTION TO S.T.P. INFLUENT
24"
1'-3"
1'-8"
2"
SECTION ELEVATION

FIGURE 21

COMBINED SEWER OVERFLOW PUMP STATION

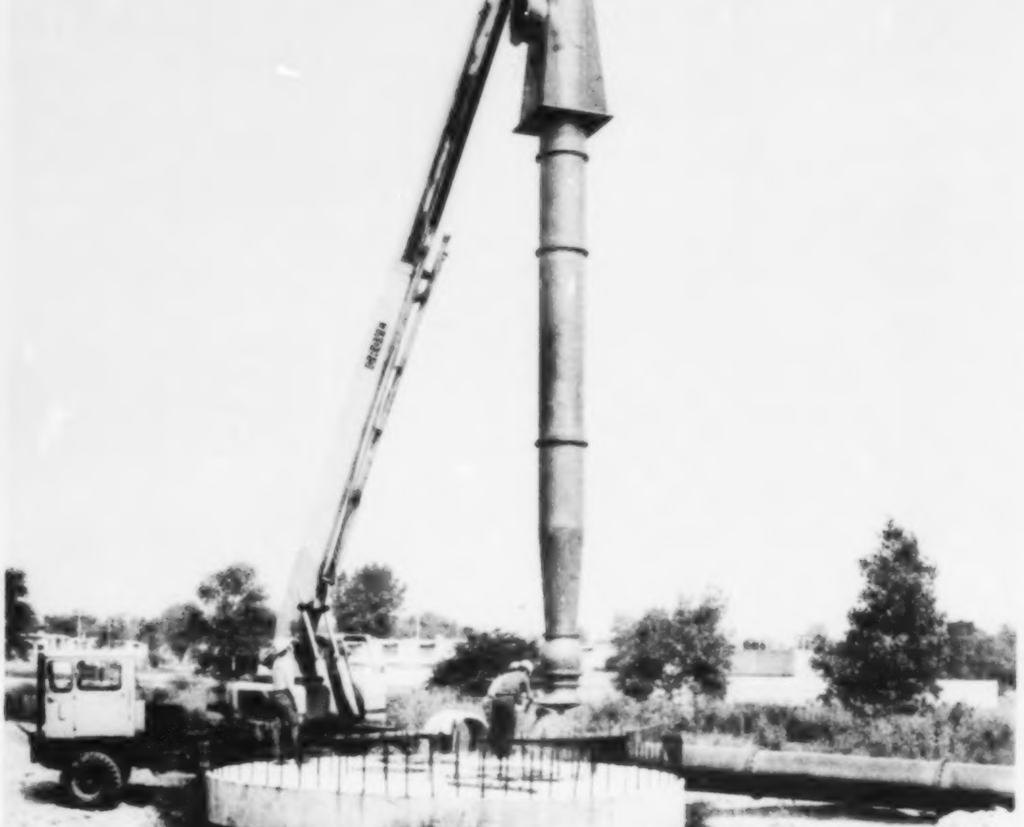


FIGURE 22. COMBINED SEWER PUMP INSTALLATION

overflow samples through the flow proportioned sampler which dipped up samples at a rate proportional to the quantity of wastewater being pumped at the time.

The sampler was located out-of-doors in a weather protected enclosure. The composite sample was held in a 5 gallon polyethylene container located in a refrigerated compartment.

The rate at which the sample dipper operated was controlled by the status of the pumps in the pump station as depicted in the control logic diagram shown in Figure 23. The output signals from the pulser that drive the pulse counter are always out of phase with each other. The pulse operates continuously. The pulser output is one pulse for each 9000 gal. of the total of the three pumps delivery.

Lakelet No. 1

Lakelet No. 1 covers an area of 1.5 acres with a volume of 0.75 million cubic feet. The accumulated volume of overflow was determined, for design purposes, by assuming that storms giving rise to a 1.5 inch accumulation would be collected. The runoff coefficient for the accumulated flow was taken as 0.6 and therefore the volume of overflow was determined to be 0.69 million cubic feet.

The runoff coefficient of 0.6 was used in this calculation as opposed to 0.5 as used previously on the basis that with a longer time of concentration, the percent of runoff would increase as the ground becomes more saturated. The lakelet was thus initially sized as follows:

$$(1.5\text{ in.})(0.6\text{ runoff coeff.})\left(\frac{1\text{ ft}}{12\text{ in.}}\right)(212\text{ acres})(43{,}560\text{ sq ft/acre}) = 692{,}604\text{ cu ft},$$

say 0.69 m cu ft.

It was considered desirable to increase the retention capacity incrementally to a value of 0.75 m cu ft. This increased capacity permitted the existence of a minimum depth of water (approximately 3 feet) in the lakelet. (The maximum depth was 17 feet.) The minimum depth had two basic functions: first, it negated the effect of groundwater pressure on the lakelet bottom such that the bottom would not be adversely affected; and second, the water would cover any possible accumulation of sludge in the bottom of the lakelet and would prevent odors from occurring.

The resulting capacity of Lakelet No. 1 provided retention for a storm giving rise to an average intensity of 0.38 in./hr for 4 hours. Any accumulated rainfall exceeding this amount would thus exceed the capacity of Lakelet No. 1 and cause an overflow into Lakelet No. 2. The storm frequency associated with this intensity and duration would be approximately a 2-year storm, based on "Probability Rainfall Frequency Curves" from the City of Detroit, Michigan. The probability curves were worked up from rainfall records of 1896 through 1942 by the City of Detroit.

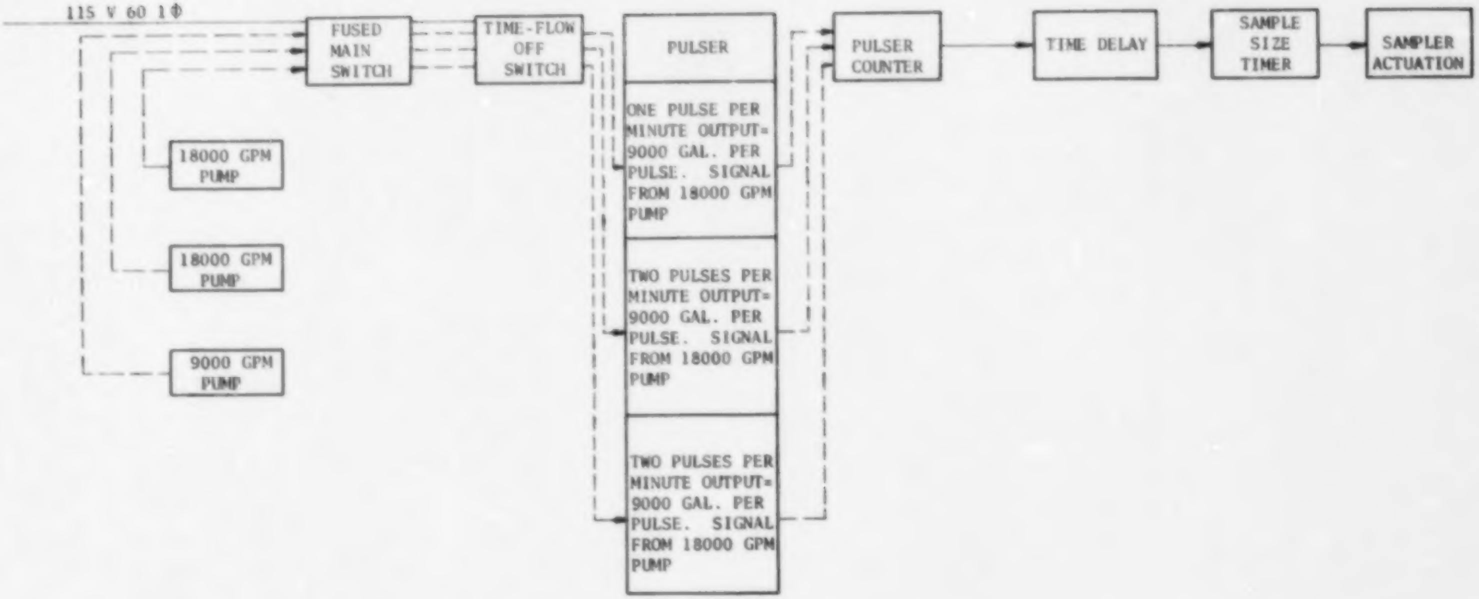

115 V 60 1Φ
FUSED MAIN SWITCH
TIME-FLOW OFF SWITCH
PULSER
ONE PULSE PER MINUTE OUTPUT= 9000 GAL. PER PULSE. SIGNAL FROM 18000 GPM PUMP
TWO PULSES PER MINUTE OUTPUT= 9000 GAL. PER PULSE. SIGNAL FROM 18000 GPM PUMP
TWO PULSES PER MINUTE OUTPUT= 9000 GAL. PER PULSE. SIGNAL FROM 18000 GPM PUMP
PULSER COUNTER
TIME DELAY
SAMPLE SIZE TIMER
SAMPLER ACTUATION
18000 GPM PUMP
18000 GPM PUMP
9000 GPM PUMP

FIGURE 23. LOGIC DIAGRAM FOR AUTOMATIC FLOW PROPORTIONED SAMPLER

If the capacity of the lakelet were exceeded, the excess would overflow into Lakelet No. 2 directly. Lakelet No.'s 1 and 2, plus the channel, each were designed to receive an extra two feet of water when Lakelet No. 1 capacity was exceeded. The extra two feet of capacity amounted to 0.5 m cu ft. Thus, the maximum capacity for capturing any overflow occurrence would be 1.19 m cu ft.

The operation of Lakelet No. 1 provided a 24 hour aeration and settling period for captured overflows prior to their uniform withdrawal, at a 1 mgd rate, for further treatment. The lakelet thus provided a varied average retention period (for different overflow occurrences) from 1 day to approximately 4 days. The maximum average detention time is calculated assuming that the entire lakelet would be dewatered as follows:

$$1 \text{ day} + \frac{(0.75 \text{ m cu ft})(7.48 \text{ gal./cu ft}) \; \frac{1}{(1 \text{ mgd})}}{2} = 3.8 \text{ day}$$

Hence, as the volume flowing into Lakelet No. 1 from any rainfall occurrence approaches zero, the minimum average detention time would be reduced to 1 day.

The lakelet was supplied with continuous aeration by mechanical floating aerators and was obscured from public view by high banks. The lakelet acted as a receiving and holding reservoir and, with its 1 to 4 day average detention time, allowed for slow treatment (settling and aeration) and uniform discharge between storms.

The combined sewer overflows entered the lakelet through a cascade inlet which was designed for some aeration and anti-erosion protection. This structure can be seen in Figure 24.

Aeration

The lakelet was supplied with two floating aerators that were moored to the banks as shown on the site plan, Figure 18. The aerators were designed to ride the surface level of the lakelet and provide continuous aeration and mixing. The aerators were positioned over 20' x 20' concrete slabs for anti-erosion protection when the lakelet was at a low level.

Floating aerators were chosen for this application because of the fill and draw operation of Lakelet No. 1. Diffused aeration could not be as effectively controlled in this application for optimum efficiency at all times. Also, even though aerobic digestion of accumulated sludge would have resulted from diffused aeration, sedimentation in the lakelet would have become impossible.

The relationship between the rate of oxygen demand and the rate of oxygen transfer (by means of aeration equipment) was not a completely finite problem due to three phenomena. One, the volume of water in Lakelet No. 1 was almost continously changing, two, the BOD was continuously changing, and three, the probable existence of an anaerobic layer at the bottom of the lakelet all resulted in widely varying oxygen

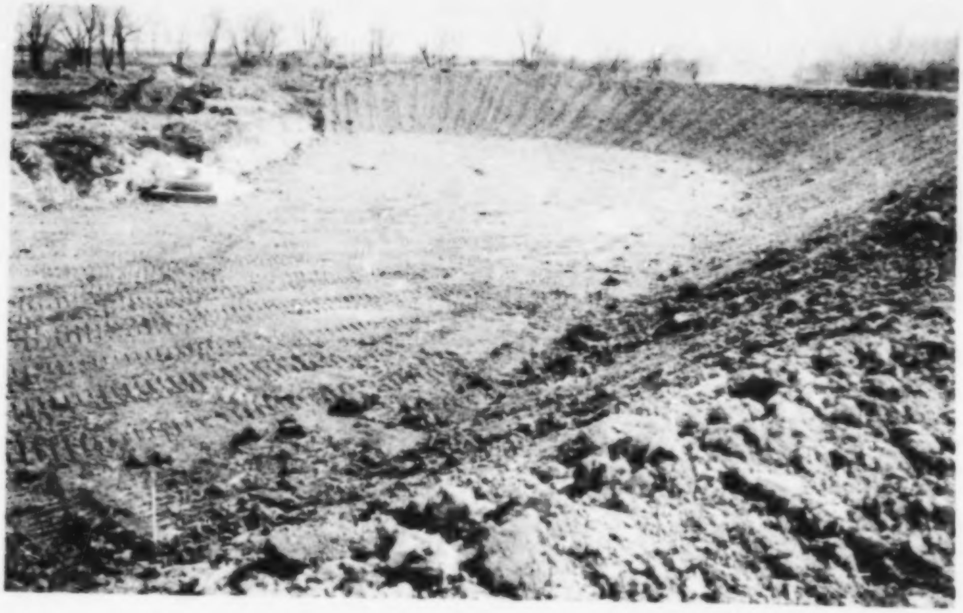


FIGURE 24. LAKELET NO. 1 DURING CONSTRUCTION

demand characteristics within the lakelet. Therefore, some basic assumptions were required.

It was assumed (and it was later confirmed) that as long as a relatively high dissolved oxygen (DO) content was maintained in the upper three feet of water from which the wastewater was drawn, an acceptable treatment performance would be experienced. The floating aerators were therefore designed to provide mixing in the upper layer of the lakelet and to supply sufficient oxygenation to maintain the relatively high DO level required.

The aerators were designed to provide aeration to a full lakelet (0.75 m cu ft) for an average detention time of 3.8 days assuming an average BOD_5 concentration of combined sewer overflows entering the lakelet to be 100 mg/l. (The 100 mg/l was the initially assumed influent concentration which was used prior to the availability of preconstruction sampling data.)

It was hence determined that the maximum oxygen uptake would be approximately 44 lbs O_2/hr. The rate of oxygen transfer was calculated as follows:

$$R_s = \frac{R_a \, (C_{sw}-C_1) 1.024^{t-20} \times \theta}{9.17}$$

where R_s = rate of oxygen transfer to wastewater, lbs O_2/hp-hr

R_a = rate of oxygen transfer under standard test conditions, lbs O_2/hp-hr

C_{sw} = saturation value of wastewater at operating temperature, mg/l

C_1 = concentration level of DO (residual), mg/l

t = operating temperature, °C

θ = ratio of oxygen transfer to wastewater to that of water.

Therefore, assuming t = 10°C (50°F):

$$R_s = \frac{3.16(11.0-1.5)0.79}{9.17} \times 0.9$$

$$R_s = 2.32 \text{ lbs } O_2/\text{hp-hr.}$$

The aerator requirement becomes:

$$\frac{44 \text{ lbs } O_2/\text{hr}}{2.32 \text{ lbs } O_2/\text{hp-hr}} = 18.96 \text{ hp} \qquad \text{use: } 20 \text{ hp.}$$

Two 20 hp aerators were therefore selected, both of which could supply more than 45 pounds of oxygen per hour, on the basis that one would always be available as a standby in the event of a malfunction in the other. The aerators used were of a type which pumped water from beneath the surface of the lakelet and discharged it radially at the surface with a high velocity impingement pattern.

The aerators were equipped with anti-erosion plates which supplied extra protection from erosion of the bottom and also supplied a support stand for the aerators to rest on when the lakelet was empty (see Figure 25).

Floating Outlet

The lakelet effluent was withdrawn through the floating outlet (the intake suspended just below the surface) shown on Figure 26 . This type of outlet was used in order to allow the drainage of the lakelet from below any surface scum layer and above the bottom sludge mass. A bottom drain was also supplied for maintenance purposes. The withdrawal of combined sewer overflows from Lakelet No. 1 was at a controlled rate of 1 mgd.

It was estimated that the 1 mgd flow rate would normally empty the lakelet in sufficient time to provide capacity for the next overflow occurrence.

Lakelet No. 1 can be seen as it looked prior to start-up in Figure 25. This figure includes the cascade inlet, aerators, and a portion of the floating outlet. Note that Lakelet No. 2 cannot be seen wrapping around the far side of Lakelet No. 1 due to the high banks of Lakelet No. 1. These banks effectively obscured Lakelet No. 1 from view from the surrounding area which was intended to aesthetically enhance the entire site.

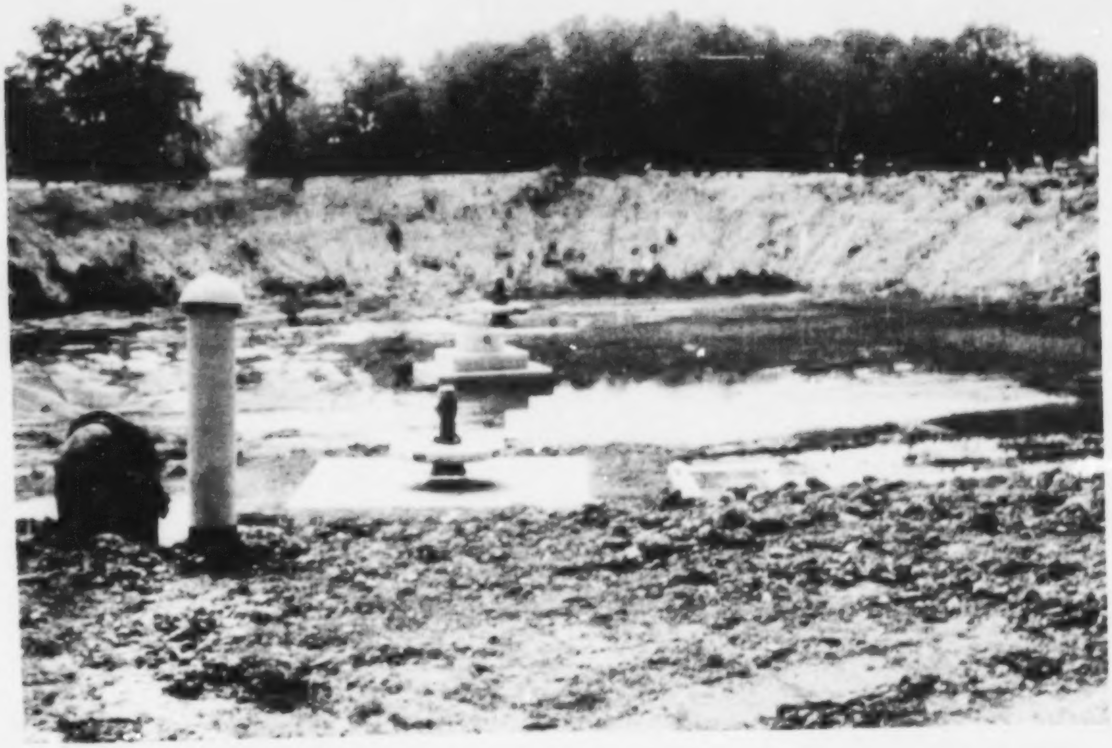


FIGURE 25. LAKELET NO. 1 PRIOR TO START-UP

Pump Station No. 1

Pump station no. 1 was a typical two pump package sewage pump station with each low head pump capable of delivering 1 mgd to the process building. The flow control system for this pump station was located in the process building (See Figure 27) such that it could be used with both pump station no. 1 and pump station no. 2 as described in this report.

A bubbler system was installed in pump station no. 1 for the purpose of controlling the pumps and indicating and recording Lakelet No. 1 water level. The indicator-recorder and the on-off-automatic switches for the pumps were located on the control panel in the process building.

Flow Control Unit

Two identical flow control units were installed in the process building; one for controlling flows entering the microstrainer and one for controlling flows from the sand filter effluent discharge.

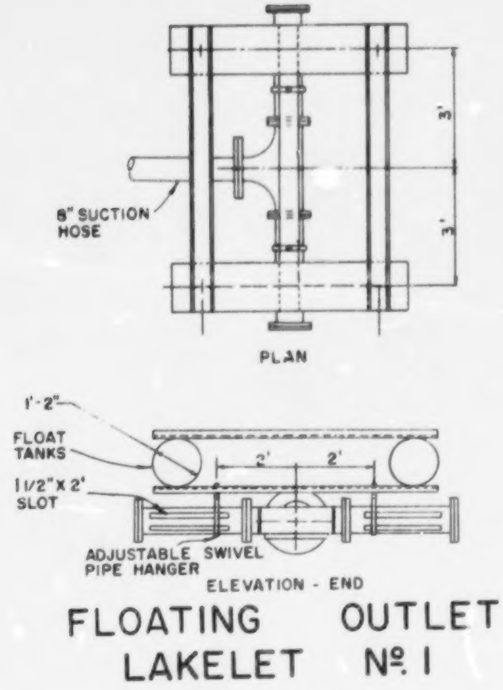


FIGURE 26. FLOATING OUTLET LAKELET NO. 1

Each flow control system consisted of a magnetic flow meter, integral converter and a tight shut-off butterfly control valve complete with integral electric motor operator and a rate setting controller. The system was capable of repeatable control with an average accuracy of ±3% over a 12:1 range.

The flow meters had polyurethane liners and stainless steel electrodes which were suitable for ultrasonic cleaning and, for maintenance purposes, the electrodes could be replaced without disassembling the meter body.

The butterfly valves were operated directly from the output of the controller. Each valve contained a solid state signal converter with adjustments for stroking the valve over the best operating range. The motor operator was burnout proof with an "auto-off-open-off-close" switch for local control.

The controller received the flow signal from the signal converter located on the magnetic flow meter and produced an output signal to the valve positioner to maintain the desired flow rate. The controller had a manual-automatic-station and valve position gauge with a 9" tape scale indicator.

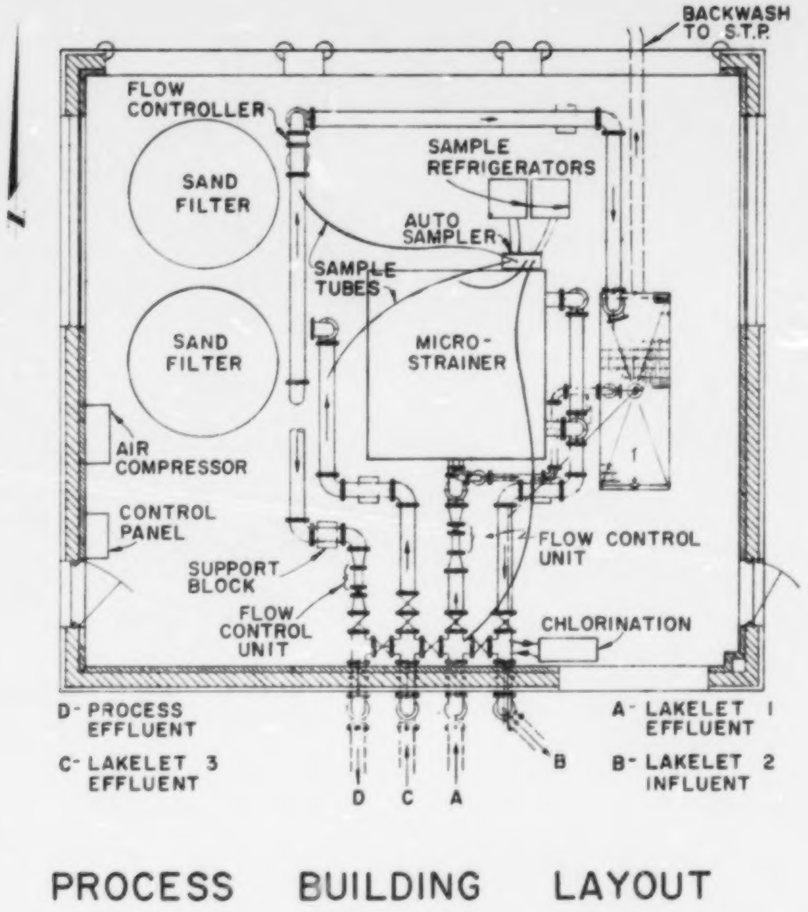

BACKWASH TO S.T.P.
FLOW CONTROLLER
SAMPLE REFRIGERATORS
SAND FILTER
AUTO SAMPLER
SAMPLE TUBES
SAND FILTER
MICRO-STRAINER
AIR COMPRESSOR
CONTROL PANEL
FLOW CONTROL UNIT
SUPPORT BLOCK
FLOW CONTROL UNIT
CHLORINATION
D- PROCESS EFFLUENT
C- LAKELET 3 EFFLUENT
A- LAKELET 1 EFFLUENT
B- LAKELET 2 INFLUENT
B
D
C
A
PROCESS BUILDING LAYOUT

FIGURE 27. PROCESS BUILDING LAYOUT

Microstrainer (Zurn Industries)

The microstrainer was located in the process building and was designed to receive flows primarily from Lakelet No. 1. However, Lakelet No. 3 flows were diverted through the microstrainer during periods of recirculation when Lakelet No. 1 was not being emptied. The design flow was 1 mgd as previously discussed.

The microstrainer design was somewhat conservative since the quality of Lakelet No. 1 effluent could not be estimated beforehand. The suspended solids concentration in the microstrainer influent was expected to vary because of the variable level in Lakelet No. 1. Hence, the opinion was reached that the microstrainer should be installed with a 60 micron screen which could later be changed to a smaller size if necessary.

The hydraulic loading was designed for 10 gal/ft^2/min, which is near the middle of the microstrainer hydraulic loading range. This criteria resulted in a microstrainer with a 6' diameter by 6' long drum that would operate approximately 60% submerged.

The filter wash water was taken from the effluent side of the mesh within the tank such that no external water source was required. The water was pumped at 36 gpm and 40 psig by a vertical turbine type pump.

A hooded ultra-violet light covered the entire length of the drum to prevent algae growths or scum build-up from occurring.

Figure 28 shows the microstrainer as installed in the process building. The spray hood, backwash pump, driving mechanism and effluent piping can be readily observed in this photo. Some changes were made concerning the unit such as the attachment of an automatic sampling unit to the microstrainer as depicted in Figure 27.

Disinfection

The chlorine-chlorine dioxide system was designed to produce data for the evaluation of the effectiveness of using chlorine and chlorine dioxide

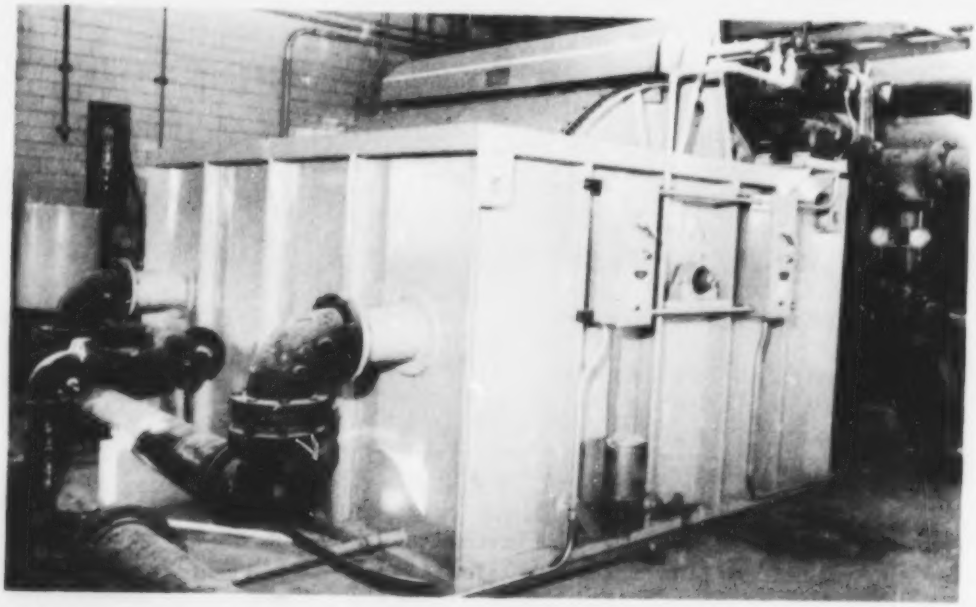


FIGURE 28. MICROSTRAINER

as disinfecting agents. Please note however, that the chlorine dioxide investigations could not be completed in the time allowed for operation of the project and thus were deleted and left for future study.

The chlorine-chlorite process was used for generating chlorine dioxide during this project. The basic reaction is:

$$2\ NaClO_2 + Cl_2 \longrightarrow 2\ ClO_2 + 2\ NaCl$$

Thus, 1.34 pounds of pure sodium chlorite ($NaClO_2$) will react with 0.5 pounds of chlorine (Cl_2) to produce 1.0 pound of chlorine dioxide (ClO_2).

Since chlorine dioxide is more potent than chlorine gas, the chlorine ejector, chlorinator, and gas supply system was designed to deliver chlorine gas according to the maximum expected chlorine demand.

Chlorine requirements vary with the type of wastewater and the degree of treatment the wastewater has received prior to chlorination. As noted by WPCF Manual of Practice No. 11, "Operation of Wastewater Treatment Plants", common values for chlorine dosage will be in the ranges shown in Table 5.[8]

TABLE 5. COMMON VALUES OF CHLORINE REQUIREMENTS (mg/1)

Type of Sewage	Demand
Raw sewage (fresh to stale)	6-12
(septic)	12-25
Settled sewage (fresh to stale)	5-10
(septic)	12-40
Trickling filter effluent (normal)	3-5
(poor)	5-10
Activated sludge effluent (normal)	2-4
(poor)	5-8

The quality of Lakelet No. 1 effluent could not be pre-determined. However, the chlorine demand was estimated to be, based on Table 5, in the range of 5 to 10 mg/1 of chlorine. Therefore, the chlorine-chlorine dioxide system that was furnished included a 0-100 PPD chlorinator, remote ejector, booster pump for ejector supply, remote meter and rate valve, chlorine dioxide generator, sodium chlorite pump, 55 gal. storage drum with angle iron stand, corporation stop diffuser, and piping of both PVC and wrought iron. Most of these components of the disinfection system can be seen as installed in the process building in Figure 29.

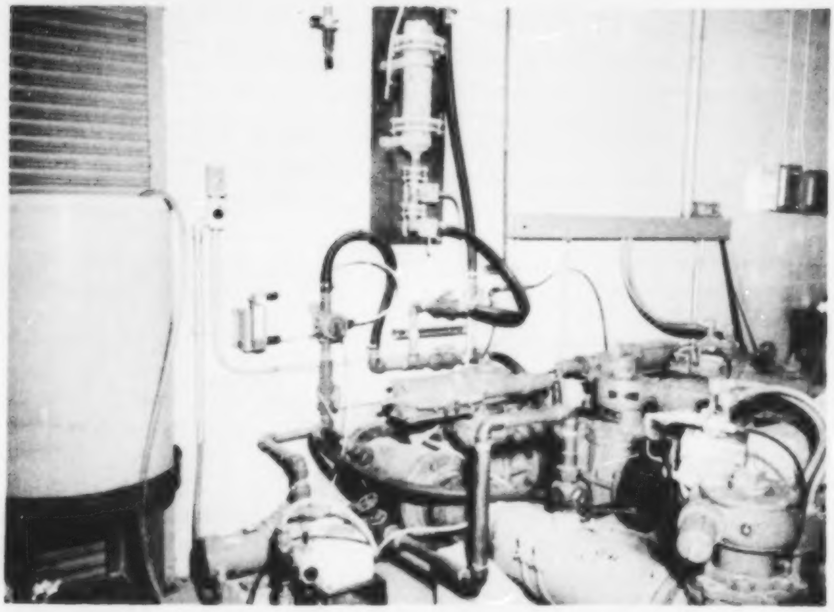


FIGURE 29. DISINFECTION SYSTEM

The chlorinator was furnished with a gas inlet heater, gas filter, protected gas pressure gauge, automatic tight shutoff valve, and an excess vacuum shutoff valve.

The booster pump supplied treated effluent to the chlorinator ejector at a 10 gpm flow rate and 60 feet TDH with a 3 foot suction lift.

The chlorine dioxide generator was a wall mounted glass reactor with porcelain rasching rings. The sodium chlorite solution pump was a diaphragm type positive displacement duplex metering pump with a 30 gallon per hour peak capacity.

Chlorine Supply System

The chlorine supply system included a pressure reducing valve, chlorine gas flow transmitter, recorder, and totalizer. The chlorine flow recorder was mounted on the Control Panel in the process building. The chlorine pressure reducing valve was a self-actuated, diaphragm type, and was mounted directly on the pipeline without additional support. The chlorine flow transmitter was an integral orifice, differential pressure transmitter (force-balance type). The chlorine flow recorder was an all solid state recorder with a 4", 30 day strip chart and a contactless feedback system. The chlorine flow totalizer was an electronic integrator with a 6 digit non-resettable counter.

Lakelet No. 2

Flows entered Lakelet No. 2 through a vertical standing inlet pipe which discharged the treated overflows at the minimum Lakelet No. 2 water elevation.

Lakelet No. 2 had a 0.1% bottom slope toward the lakelet drain and 1 vertical to 2 horizontal side slopes from the bottom to the high water elevation. 1 vertical to 4 horizontal side slopes were constructed from the high water elevation to the appropriate site grade. The lakelet was constructed with a 17" clay liner for the prevention of exfiltration.

The lakelet had a 1.1 m cu ft capacity and covered 2.8 acres. It supplied 8 days detention time at the normal 1 mgd flow rate and was designed to provide initial chlorine contact time for the overflows plus natural surface aeration and photosynthetic oxygenation.

The lakelet emptied into the channel which conveyed the treated overflows to Lakelet No. 3. The channel was 25 feet wide and 5 feet deep and created a transportation barrier between the park side of the site and the existing sewage treatment plant side of the site.

Lakelet No. 3

Lakelet No. 3 had a 0.93 m cu ft capacity and covered an area of 2.3 acres. It provide 7 days detention time at the 1 mgd rate and also provided oxygenation by natural and mechanical surface aeration plus photosynthetic oxygenation.

The mechanical surface aerator used in Lakelet No. 3 was identical to those used in Lakelet No. 1. This aerator was supplied as a safety factor in case Lakelet No. 1 overflowed into Lakelet No. 2 and created too large an oxygen demand for the natural surface aeration and photosynthetic oxygenation to meet. The aeration capacity of the aerator was designed to be equal to the aeration capacity of those aerators used in Lakelet No. 1 primarily as a convenience factor.

Pump Station No. 2

Lakelet No. 3 could be emptied only through pump station no. 2, a three pump package sewage pump station. The pump station contained two high head pumps for delivering flows to the sand filters and one low head pump for delivering flows to the microstrainer for recirculation.

The pump station was also equipped with a bubbler system to indicate the Lakelet No. 3 water level. The indicator-recorder for the bubbler system was located on the control panel in the process building.

Pressure Sand Filter (Baker Filtration)

The water from Lakelet No. 3 was pumped by pump station no. 2 onto the pressure sand filter, located in the process building, prior to discharge into the Clinton River. The sand filter was an automatic high-rate two stage series type pressure sand filter.

The filter was designed to receive flows through the first tank, which had a coarse sand media, and then through the second tank, which had a somewhat finer media. The filter tanks were backwashed by using water filtered through one tank to backwash the other.

The design flow rate for this sand filter was 15 gpm/ft^2. Each filter tank was 90 inches in diameter with a side shell height of 54 inches. The filter media consisted of uniformly graded silica sand 30" deep which was free from limestone and clay. The primary filter contained media of effective size 0.45 mm and the secondary filter contained media of effective size 0.30 mm; both media grades had a uniformity co-efficient not greater than 1.4. A support grade Garnet Sand 3" deep of effective size 1.45 mm was provided in both the primary and the secondary filters.

The filtration rate was controlled by a flow control unit located on the filter effluent line as previously described. The backwash flow rate was controlled by an orifice type flow controller set at 665 gpm.

The sand filter was set up to operate automatically during the course of the project. The bubbler system in pump station no. 2 sensed the Lakelet No. 3 water level which controlled the automatic start and stop of the high head pumps in the pump station. The sand filter backwash system was actuated by either a preset time delay (typically 8 hours) or an inordinate pressure drop (typically 58 feet of water) between the filter influent and filter effluent lines.

The flow pattern through the filter was controlled by a series of pneumatic cylinder actuated butterfly valves. The valve sequence was controlled by pneumatic time delay relays and timers located in the sand filter control panel.

Figure 30 shows the sand filter as installed in the process building. The vertical pipe on the far right of the photo conveyed the filter backwash to the sewage treatment plant.

Control Panel

Figure 31 shows the control panel as installed in the process building. The control panel was installed with three strip chart recorders, two flow controllers, three flow totalizers, and the various control switches required to manually operate the facility.

One strip chart recorder was used to record the variation of Lakelet No. 1 and Lakelet No. 3 water levels, one recorded the pumpage from Lakelet Nos. 1 and 3, and the other recorder kept track of the chlorine gas flow. The flows from the lakelets and the chlorine tanks were also totalized on the control panel.

The two flow controllers were normally set at 1 mgd (700 ± gpm), but could be varied from 100 gpm to 900 gpm.

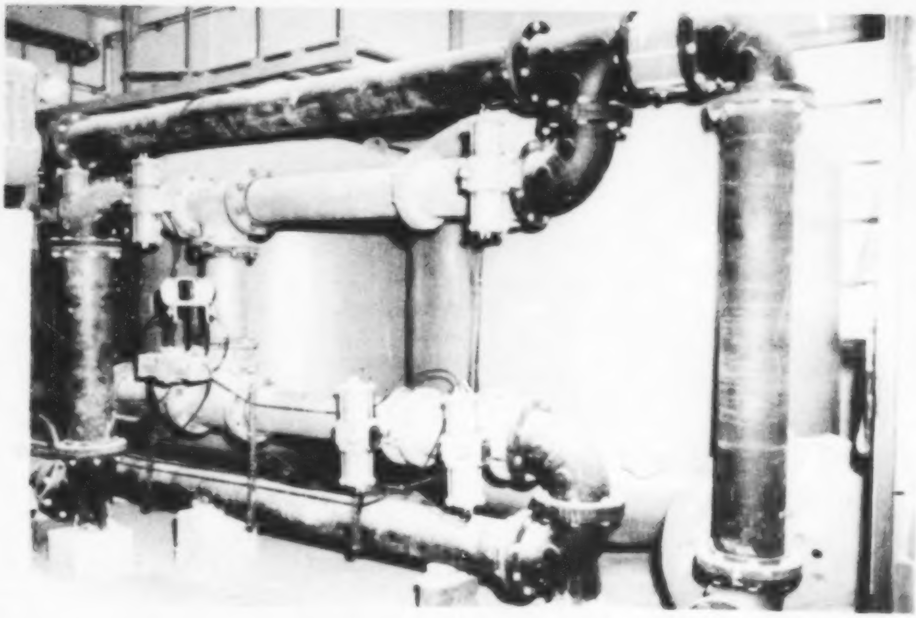


FIGURE 30. SAND FILTER

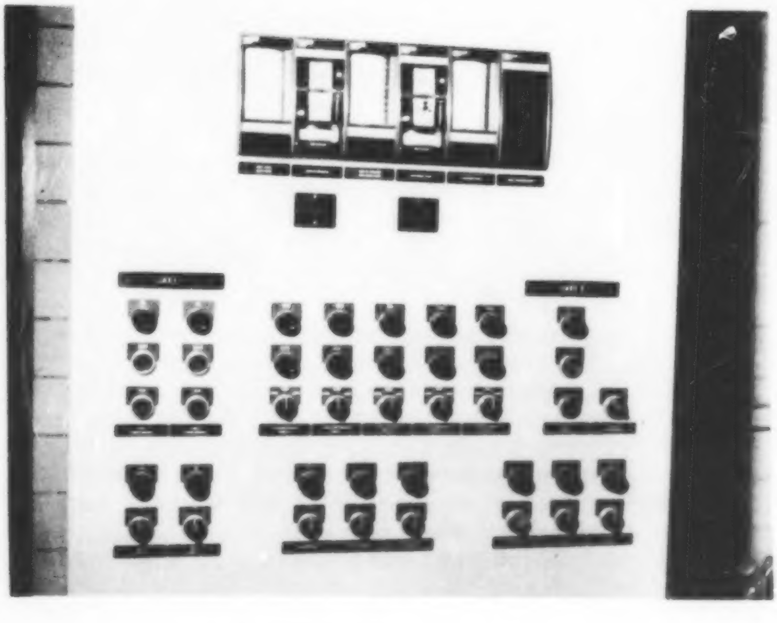


FIGURE 31. CONTROL PANEL

The three surface aerators had push button start and stop buttons while the remaining controls were hand-off-automatic switches.

Automation of Treatment Facility

All process equipment including pump stations was designed to operate automatically once the combined sewer overflows reached the combined sewer overflow pump station.

CONSTRUCTION COSTS

The total construction cost of this facility was broken down into grant eligible and grant non-eligible costs. The grant non-eligible costs were those which did not pertain to the treatment facility such as the interceptors, overflow chambers, etc. Table 6 shows the breakdown of the construction costs. These costs do not include those for engineering, legal, administrative, land acquisition, bonding, and contingencies.

TABLE 6. CONSTRUCTION COST BREAKDOWN

Items	Construction cost	Eligible	Non-eligible
Section A (pre-construction monitoring facilities)	\$ 58,995.00	\$ 58,995.00	
Section B (project construction)			
Contract 1A (interceptors, overflow chambers, drainage)	226,872.70	162,391.00	\$ 64,481.70
Contract 1B (lakelet excavation, grading)	166,320.00	166,320.00	
Contract 2 (process equipment[a], building modifications)	249,715.60	249,715.60	
Total	\$701,903.30	\$637,421.60	\$ 64,481.70

[a] Although Contract 2 was a lump sum contract, the following itemized breakdown might be of assistance:

Overflow Pump Sta.	\$43,000	Pressure Filter	\$89,780
Automatic Sampler	3,600	Cl_2-ClO_2 (incl. instrumentation)	23,000
Aerators	16,000	Electrical Work	50,735.60
Microstrainer	23,600		

SECTION VI

POST CONSTRUCTION STUDIES

OBJECTIVES

The post-construction studies were performed between July 20, 1972 and April 15, 1973. The post-construction studies aimed at demonstrating the concept of treating combined sewer overflows in a Treatment-Park Site to provide pollution abatement, recreation, and open space, and incidentally to utilize some of the process water for irrigation purposes Specifically, the studies were conducted in the following areas:

- Treatment of combined sewer overflows
- Effect of overflows on river quality
- Recreation at Treatment-Park Site
    - Park facility
    - Boating and fishing
- Open space and transitional land use
- Treatment of overflows vs. separation program
- Multi-benefit uses of public funds

As the sufficiency of the treatment affects the acceptability of the lakes (or process water) for use in a spray irrigation system and use as a recreation facility, a major emphasis during the course of the studies was placed on the evaluation of the proposed treatment concept. Each unit process was carefully observed with this purpose in mind.

The sampling program was designed to produce sufficient data for the evaluation of unit processes and treatment efficiencies. The sampling locations were designated as shown in Table 7.

Table 8 shows the sampling program and analyses to be performed at each location. The sampling frequency was limited since the existing personnel at the sewage treatment plant were to operate this facility. These procedures were set up primarily as guidelines for the operating personnel with deviations from the schedule to be initiated by the research staff when warranted by the project status. All data collected during this sampling program is presented in the appendix. Samples were analyzed according to "Standard Methods for the Examination of Water and Wastewater including Bottom Sediments and Sludges".[9]

The planned recreational facilities at the treatment-park site were to be scenic walks, park benches, and picnic areas. Limited water-oriented sports in Lakelet No. 3 and in part of Lakelet No. 2 were proposed to include small boat sailing, canoeing, and fishing. The discussion of this study area can be found in Section VIII of this report.

TABLE 7. LIST OF PROJECT SAMPLING LOCATIONS

Sampling location	Point designation	Description
Overflow	1, 2, 3	Overflows discharged directly to the Clinton River (magnetic flow meters and automatic samplers)
Treatment-Park Site	4	Combined Sewage Overflow Pump Station (automatic flow-proportioned sampler)
	5	Lakelet 1 Effluent - Microstrainer Influent
	6	Microstrainer Effluent - Lakelet 2 Influent
	7	Lakelet 2
	8	Lakelet 2 Effluent
	9	Lakelet 3 Effluent
	10	Sand Filter Effluent prior to discharge to the Clinton River
Clinton River	A	Immediately Upstream of the Reach Affected by the Demonstration Project
	B	Immediately Downstream of the Reach Affected by the Demonstration Project

FACILITY UNITS OPERATION EVALUATION

This section of the report deals with unit operations and their evaluation with respect to operational efficiency. Those facility units which performed as predicted are not discussed in this section. Only those units which are considered to warrant mention are discussed.

Overflow Chambers

The U-trough method for intercepting combined sewer overflows proved to be an efficient and economical approach to the collection problem. Figure 32 shows the Clemens Street overflow structure during a peak dry weather flow condition. If a rainfall had occurred at this time, the U-trough would have overflowed into the interceptor allowing only a minimum surcharge to be conveyed to the treatment plant.

TABLE 8. SAMPLING PROGRAM AND ANALYSES

	SAMPLING LOCATIONS											
	1	2	3	4	5	6	7	8	9	10	A	B
Flow Condition												
Storm	X	X	X	X	X	X	X	X	X	X	X	X
Recirculation thru Microstrainer					X	X		X	X		X	X
Recirculation Bypassing Microstrainer							X	X	X		X	X
Sampling Techniques												
Composite Sample (Flow Proportioned)				X								
Composite Sample (One/24 Hrs.)					X	X		X	X	X		
Grab Sample (One/4 Hrs.)	X	X	X									
Grab Sample (One/24 Hrs.)							X				X	X
Grab Sample (2 to 3/Week)					X		X	X	X	X	X	X
Analysis												
SS	G	G	G	C	C	C		C	C	C	G	G
VSS	G	G	G	C	C	C		C	C	C	G	G
pH	G	G	G	C	C	C		C	C	C	G	G
PO4	G	G	G	C	C	C		C	C	C	G	G
NO_3-N	G	G	G	C	C	C		C	C	C	G	G
NH_3-N	G	G	G	C	C	C		C	C	C	G	G
BOD	G	G	G	C	C	C		C	C	C	G	G
Total Coliforms							G	G	G	G	G	G
Fecal Coliforms							G	G	G	G	G	G
Oil				C						G		
DO					G		G	G	G	G	G	G
Cl_2							G					

C - Composite Sample

G - Grab Sample

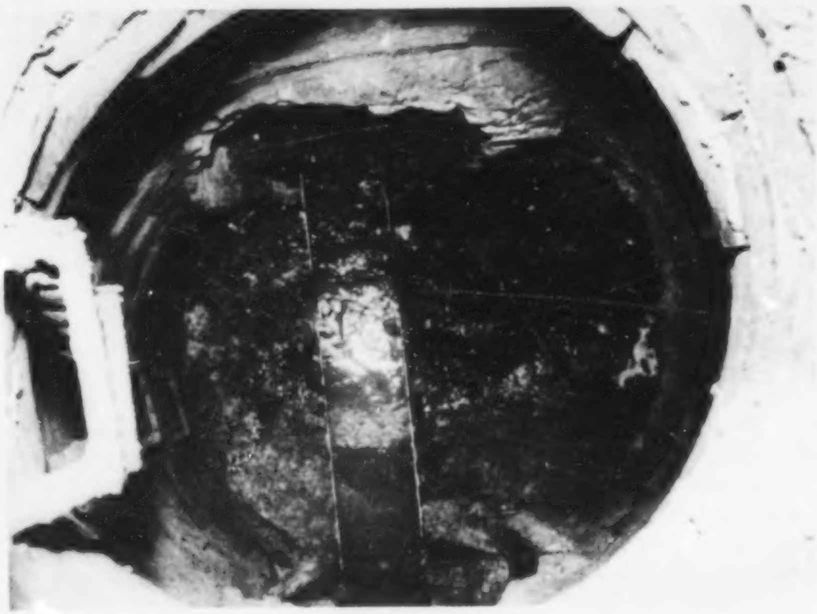


FIGURE 32. CLEMENS STREET OVERFLOW STRUCTURE (DURING CONSTRUCTION)

By constructing the U-trough and drop manhole such that the overflows would be intercepted ahead of the existing diversion chambers, the diversion chambers could be left alone to divert the dry weather flow to the Sewage Treatment Plant and remain as an emergency overflow outlet in case of a combined sewage overflow pump station failure. It is believed that the overflow chambers were used once during post-construction studies when a power failure knocked out the site electrical supply including the main pump station. Without power to the site, the overflows to the river could not be monitored.

In designing a U-trough for this application, two basic considerations are suggested. First, the U-trough should be sized to carry only the peak dry weather flow in order to minimize the surcharge to be conveyed to the sewage treatment plant. Secondly, the hydraulic gradient of the combined sewage overflow interceptor should be below the top edge of the U-trough since, when this condition exists, only the maximum capacity of the U-trough will be conveyed to the sewage treatment plant during design conditions.

Combined Sewage Overflow Pump Station

A "two year" design storm was experienced over the service area during the course of the project, but the full 100 cfs capacity of the pump station was not required for handling the flows. This seemed to indicate that the run-off coefficient of 0.5 was somewhat conservative. No mechanism was provided for monitoring the volume of overflows pumped into Lakelet No. 1 other than monitoring the water level of Lakelet No. 1 and recording the pumpage out. Future research projects however should provide direct measurement capability.

Problems did occasionally occur with the electrode controls when grease and other foreign materials worked their way inside the galvanized pipe and adhered to the electrodes. The electrodes then had to be removed and cleaned because of proper pump actuation being fouled by the foreign material which either caused starting the pumps too soon or not starting them at all. When the electrodes were kept clean, no problems occurred, hence, this became a regular maintenance procedure. In the city-wide project discussed in Section IX, the main pump station will have the bubbler system for pump actuation.

Flow Proportioned Sampler

Minor problems were experienced during the operation of this sampler when the weir box effluent pipe was found to be too small to handle the flows and thus the unit spilled an excessive sample into the sample container.

Clogging of sample pipes was a recurrent problem which could not be solved due to the inconsistent characteristics of the liquid being sampled. Therefore, continuous checking and periodic cleaning of the sample lines were necessary.

Cascade Inlet

The cascade inlet structure of Lakelet No. 1 proved to be very worthwhile As shown on Figure 33, the cascade inlet effectively dissipated the velocity of the wastewater entering Lakelet No. 1 and supplied some aeration. A small amount of erosion was seen around the perimeter of the structure which indicated that excessive erosion of the clay liner around the inlet would have caused a major problem if such a structure had not been incorporated.

Aerators

Lakelet No. 1 was designed to be the major treatment unit of the facility. Therefore, it was important that all processes within the lakelet be kept in full operation at all times.

The control sequence was set up to maximize dissolved oxygen concentrations and mixing of the upper layers of water. The north aerator (the one closest to the floating outlet) was shut off during the dewatering operation to allow for maximum settling to occur prior to discharge through the floating outlet. Both aerators were in operation during the collection

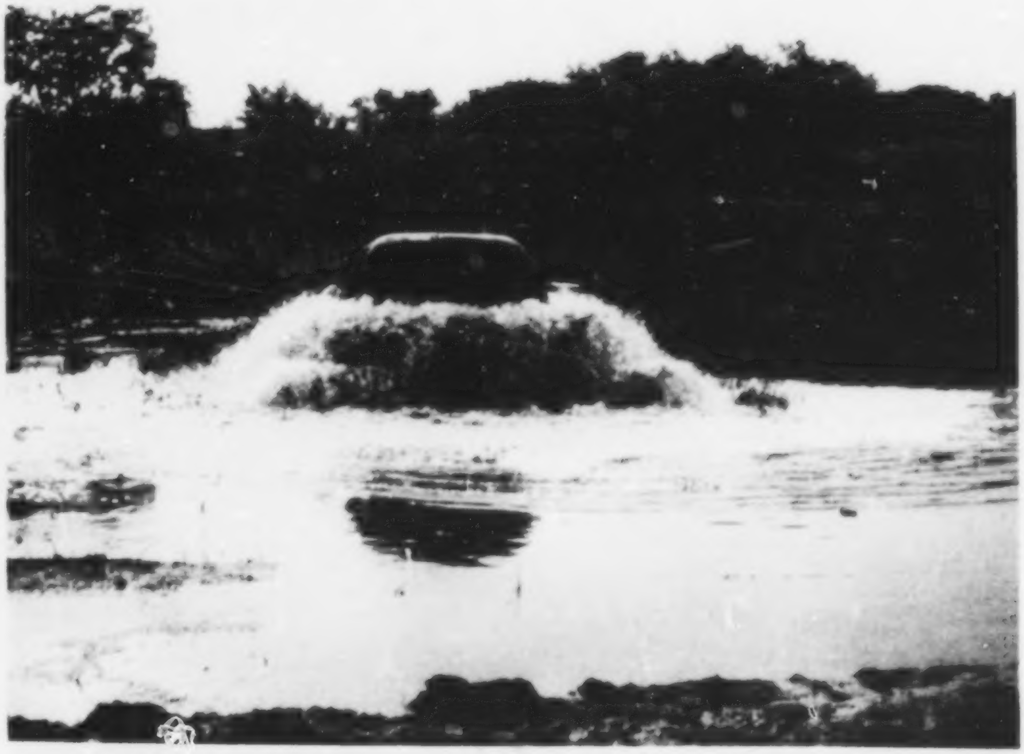


FIGURE 33. CASCADE INLET - INITIAL FLOW

(filling) operation and the 1 day detention time prior to initiation of dewatering in order to facilitate mixing and to insure that no stagnant areas developed. Figure 34 shows both aerators of Lakelet No. 1 in operation.

Each aerator was found to be capable of supplying sufficient aeration to the collected wastewater as indicated by the dissolved oxygen concentration determinations run on samples from the microstrainer influent sampling location. These concentrations were consistently near a maximum value.

Winter operation of the aerators in Lakelet No. 1 proved to be a difficult maintenance problem. It was not originally planned to run the aerators during the winter, but the plant personnel gained some valuable experience in attempting to keep them operational. Generally, the project was shut down during the winter months, due to the lack of overflows for treatment, and the Lakelet No. 1 was maintained at a minimum level.

When a winter overflow did occur, the lakelet filled and the aerators operated automatically as designed. The problem condition came into

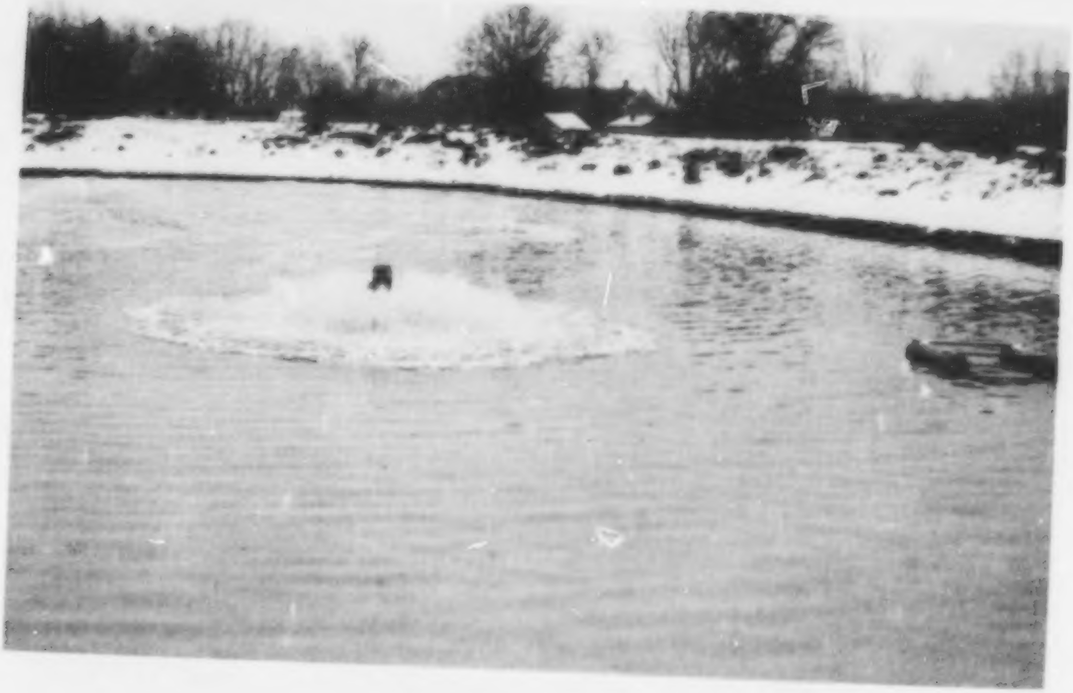


FIGURE 34. LAKELET NO. 1 IN OPERATION

effect when the temperature dropped below freezing before the dewatering operation could empty the lakelet. Thus, the lakelet would start to freeze since the aerators could not keep the water surface from freezing over the entire lakelet. Figure 35 shows one of the aerators operating as the lakelet was freezing in around it. This figure also shows the floating outlet in the ice. However, it was found that the intake, which was several inches under the surface, was open so the ice was not a problem here. The water being pumped by the aerator created substantial wave action near the aerator which tended to pile up the ice, as seen in the foam like appearance of the ice adjacent to the water.

The next circumstance which occurred was the icing up of the aerator. The fine mist that was formed by the aeration operation would freeze and adhere to the motor casing of the aerator. This situation was extremely critical since it made the already somewhat top heavy aerator even more unstable. If this situation were permitted to continue, the aerator soon would become exceedingly top heavy and would capsize. This did occur with the Lakelet No. 3 aerator.

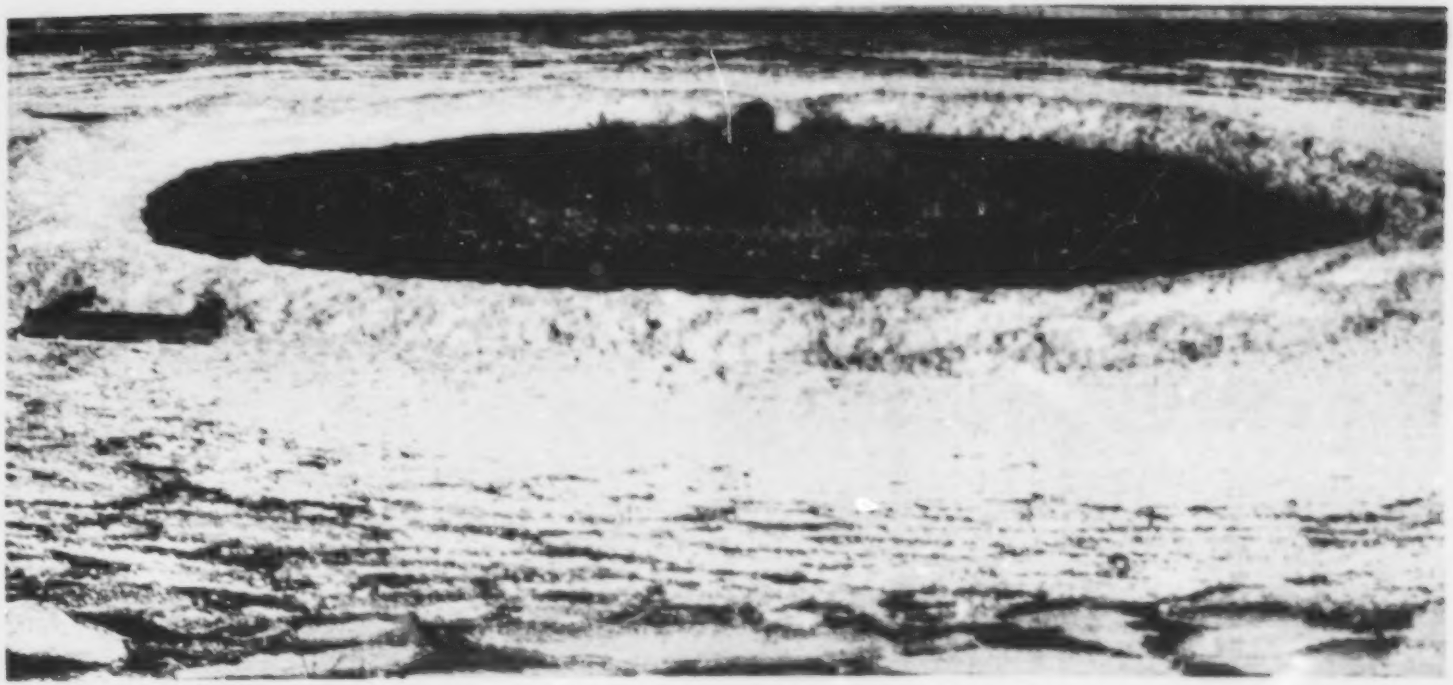


FIGURE 35. LAKELET NO. 1 AERATOR INITIAL FREEZE

Figure 36 shows one of the aerators as it was icing up. Note that the ice accumulation on the aerator impaired the ability of the aerator to throw water into the air. This condition substantially reduces the aeration and mixing capacity of the aerator.

On one occasion the aerators in Lakelet No. 1 did freeze while the lakelet was approximately half full. Upon continuing the dewatering of the lakelet, the mooring cables were also pulled loose from the sheet pile anchors.

No foolproof method has been found to prevent the icing of aerators during winter operation. Moreover, it is questionable whether winter aeration is actually required for combined sewer overflow retention facilities. The high DO levels naturally present during cold periods may be sufficient to prevent odors from occurring. Also, the rate of biological activity for treatment in this temperature range suggests that sedimentation be the only Lakelet No. 1 function strived for during winter operations.

Summer operation of the aerators presented only one problem. The aerators in Lakelet No. 1 would tend to collect pieces of cloth, weeds, and other such foreign material and wrap it around either the impeller or the erosion plate which resulted in restricting the intake. This would put an extra load on the motor which would sometimes overload the aerator enough to make it shut down. This problem was handled by the installation of reversing switches for each aerator which enabled the operator to easily unplug the aerators.

Lakelet No. 1

If Lakelet No. 1 were to be operated on a long term basis, some method for sludge removal would be required. At the end of the study period for this project, an average 9 inches of sludge accumulation was observed on the bottom of the lakelet. Anaerobic (and some aerobic) decomposition of the sludge mass was occurring as expected and the rate of increase in volume of the sludge mass, it was estimated, would have warranted a sludge removal operation on an annual basis.

Hydrographs were not constructed for Lakelet No. 1 to see when the retention capacity would be exceeded. Investigations in this area were left for the city wide project. However, from operating records, the capacity in Lakelet No. 1 was sufficient to capture each rainfall occurrence. The lakelet did overflow into Lakelet No. 2 due to a tremendous inflow of river water into the system. This overflow seemed to have no ill effects on the performance of Lakelet Nos. 2 and 3.

Regarding DO levels in the lakelet, near maximum concentrations were found to be present at all times. The samples were taken at the microstrainer influent which means that the DO levels at the submerged outlet were consistently high. A DO profile was never performed for the lakelet; however, the presence of the sludge mass on the bottom would suggest anaerobic conditions at lower levels.

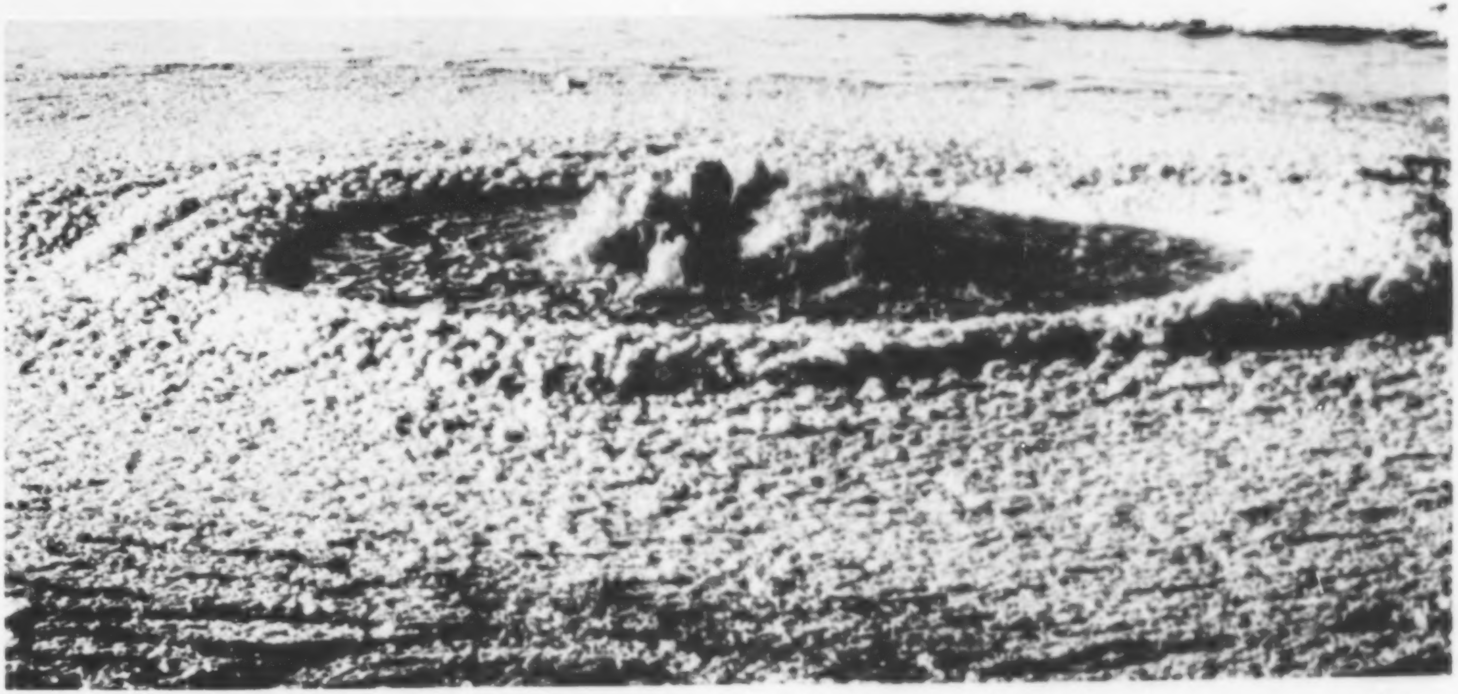


FIGURE 36. LAKELET NO. 1 AERATOR WITH ICE ACCUMULATION

Reproduced from best available copy.

Microstrainer

The 60 micron screens were kept in operation until mid-September 1972 when it was decided that the removals achieved by this unit were not satisfactory, as indicated by the data presented in Appendix B-1. At that time, 20 micron screens were installed on the microstrainer in an attempt to produce more significant removals with the unit. The screens were a molded plastic panel with 20 micron screen apertures.

Several operational problems were experienced with the microstrainer after installation of the 20 micron screens. Of major concern was the fact that the microstrainer could no longer handle the 1 mgd flow rate and the removal efficiency of the unit was not significantly increased.

The backwash pump was capable of producing a backwash water pressure of only 40 psi. While this pressure was sufficient to drive the trapped particulate material out of the 60 micron screens it was not sufficient to do so for the 20 micron screens. The 20 micron panels would thus tend to blind and prevent the full 1 mgd flow rate from being processed. This situation would then create a larger than normal differential across the screens which would tend to blind the screens even more.

Moreover, oil entering the microstrainer from Lakelet No. 1 also produced a serious problem when the 20 micron screens were in use. When a certain quantity of oil was pumped onto the microstrainer, the screens would immediately blind and the influent would begin to bypass. The only way to clear the screens was to take the unit out of service and wash the screen with an industrial strength detergent by means of a high pressure sprayer.

The automatic bypass within the microstrainer was not designed for 100% capacity. The microstrainer studies indicate however that 100% bypass capacity should be incorporated in the design.

The general operation of Lakelet No. 1 (sedimentation, aeration, etc.) and the occasional occurrence of oils in the lakelet effluent would indicate that this is not an appropriate application for a microstrainer. However, the inability of the microstrainer to perform an acceptable function had no detrimental effects on the project. The higher than anticipated degree of solids removal in Lakelet No. 1 produced a Lakelet No. 2 influent of high quality as originally predicted.

Chlorination

It was considered much more economical to install the booster pump and use process water instead of potable water, for chlorination purposes. The periodic cleaning of the coarse screen filter on the booster pump intake was not thought to be a difficult, or time consuming, maintenance chore.

Chlorination studies were initially proposed to be run for the purpose of evaluating chlorine dioxide as a better alternative for treating combined sewer overflows as compared to standard chlorine gas disinfection. Chlorine gas studies were run and a sufficient quantity of data was collected pertaining to dosing rates, costs and amounts of residual chlorine achieved. However, due to a lack of rainfall during the study period, a sufficient quantity of data was not collected for the chlorine dioxide studies to allow for any conclusive evaluation of the two alternatives. The future city-wide project proposes the use of chlorine dioxide as a method of disinfection such that further studies can be performed.

Lakelet Nos. 2 and 3

Figure 37 shows Lakelet No. 2 at the influent location.

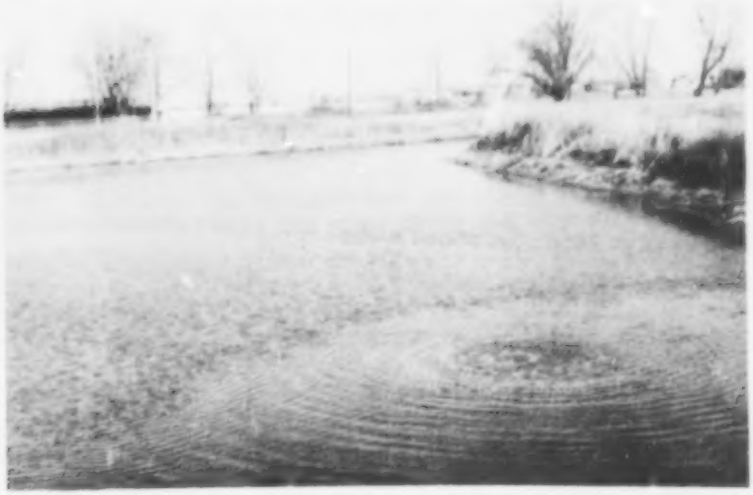


FIGURE 37. LAKELET 2 AT INFLUENT LOCATION

The lakelets were clay lined for the purpose of preventing exfiltration. The lakelets retained the water but the clay caused a significant amount of turbidity. This situation could have been rectified by the installation of a crushed stone lining; which incidentally was a part of the original design but was omitted because of the limited budget.

Sand Filter

The sand filter functioned very well during the entire project. The quality of effluent from the filter was consistent (see Appendix B-1) and no major operational problems occurred.

The most noteworthy factor concerning the application of a sand filter for this project was that the unit could be discontinued from service for any period of time and then restarted with no special maintenance procedures being required. Such is not the case, for example, with a microstrainer which requires extensive cleaning of the screens when it is taken out of service.

Automation of Treatment Facility

During the one year study period less than 20 percent of the storms occurred during a regular, 8 hour day shift. This indicates that automation of the treatment facility is a necessity. Automation less than 100 percent would have required a 3 shift per day type of operation.

Cost of Operation

The cost of operating and maintaining this facility was broken down into five categories and recorded for the period of April 15, 1973 to April 15, 1974. This cost thus represents a one year total operational cost for the combined sewer overflow collection and treatment facility. This cost includes only the operation of the treatment facility, as no park facility, as described elsewhere in this report, was developed at the time. We believe this cost would not have changed by 10 percent if the 'study year' would have been different, i.e. if more or less number of overflows had occurred.

This project was operated by city personnel from the adjacent sewage treatment plant. This situation provided operating personnel for the project as they were needed, but usually one operator was assigned to the project full time. The costs for personnel as shown in Table 9 are therefore based on an average time spent on the project by each attendant for the one year period. The costs shown in Table 9 indicate the total amount spent on each item for the one year period.

Automation of the treatment facility was estimated to cost $40,000. If the facility would not have been automated, the personnel costs would have been $62,900 instead of $32,900.

Projected operation and maintenance costs for the city wide project are presented in section IX of this report. The projection is based on a full scale operation where partial costs such as the 1/2 superintendent would not be possible.

TABLE 9. OPERATION AND MAINTENANCE COSTS

Item	Cost ($)
Personnel (includes fringe benefits)	
. 1 1/2 operators	21,000
. 1/2 superintendent	11,900
Operation & Maintenance (other than personnel)	
. materials, parts	7,500
Chemicals (chlorine)	1,300
Laboratory Supplies	1,200
Power (electricity)	2,100
TOTAL	$45,000

TREATMENT CONCEPT EVALUATION

Post-construction studies were performed from April 15, 1972 to April 15, 1973; and during this period, the facility was in full operation a majority of the time. The facility was shut down once during a repair operation of a dike failure between Lakelets 1 and 2 (an old abandoned pipe was discovered) and again during the repair of damaged underground electrical wiring. It was felt that these two shut downs did not significantly interfere with the project study program.

Statistical Analysis of Data

The post-construction studies sampling data is given in Table B-1. The post-construction operations, the river sampling data, and the rainfall data are given in Tables B-2, C-1, and C-2 respectively.

The sampling data in Table B-1 was statistically analyzed, and the analysis is given in Table 10. The parameters chosen were SS, VSS, BOD_5, PO_4-P, and NH_3-N. Lakelet No. 1 influent (Sampling Location No. 4) had a flow weighted mean SS value of 239 mg/l with 95% confidence level of ±83 mg/l. The mean BOD_5 was 75 mg/l with 95 percent confidence level of ±25 mg/l. The final effluent (Sampling Location No. 10) had a mean SS value of 14 mg/l with 95 percent confidence level of ±2 mg/l. The mean BOD_5 was 6 mg/l with 95 percent confidence level of ±1 mg/l. Figures 38 and 39 show the treatment efficiency for BOD_5 and SS respectively, as a function of the detention time through the lakes system. Both figures indicate that there is practically no improvement in the effluent quality after a detention time of approximately 12 days.

TABLE 10. STATISTICAL ANALYSIS OF POST-CONSTRUCTION STUDIES SAMPLING DATA (mg/1)

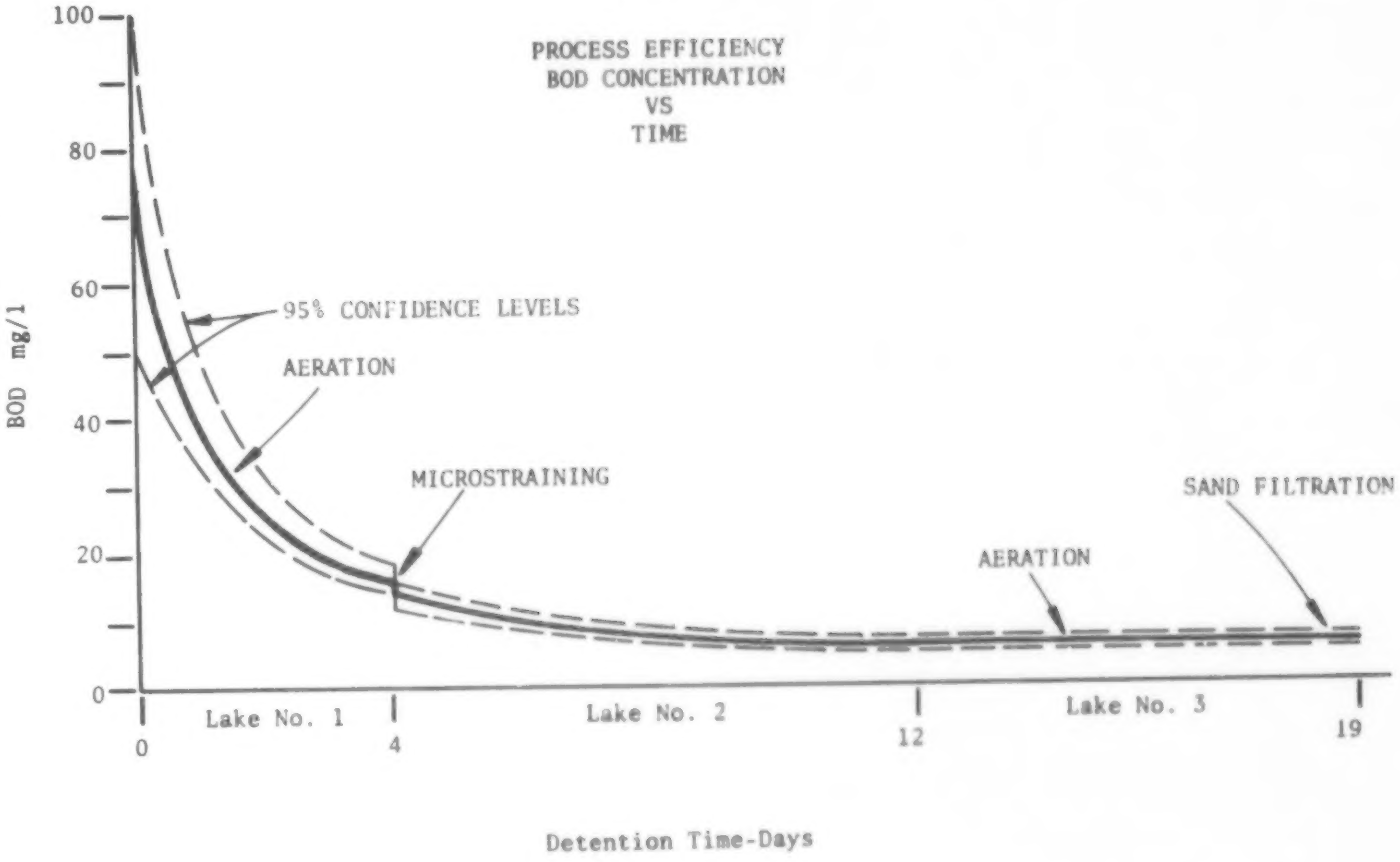
Location	Parameter	Count	Minimum	Maximum	Mean	Standard deviation	95% confidence levels
4	SS	37	20	1024	239	256	239±83
	VSS	37	10	288	79	73	79±24
	BOD_5	35	11	420	75	74	75±25
	PO_4-P	36	0.3	9.8	3.4	1.9	3.4±0.6
	NH_3-N	37	0.2	33.0	8.6	7.2	8.6±2.3
5	SS	157	10	288	57	50	57±8
	VSS	158	1	101	18	12	18±2
	BOD_5	151	2	87	16	12	16±2
	PO_4-P	154	0.2	16.0	2.7	1.5	2.7±0.2
	NH_3-N	157	0.5	32.0	7.5	5.2	7.5±0.8
6	SS	157	3	286	48	47	48±7
	VSS	157	0	56	14	9	14±1
	BOD_5	152	1	65	14	11	14±2
	PO_4-P	154	0.2	15.5	2.5	1.5	2.5±0.2
	NH_3-N	156	0.5	30.0	7.2	5.0	7.2±0.8
8	SS	32	2	59	26	17	26±6
	VSS	32	1	25	8	5	8±2
	BOD_5	30	1	14	6	3	6±1
	PO_4-P	32	1.0	5.9	2.4	0.9	2.4±0.3
	NH_3-N	32	0.9	14.5	5.4	2.7	5.4±0.9

TABLE 10 (continued). STATISTICAL ANALYSIS OF POST-CONSTRUCTION STUDIES SAMPLING DATA (mg/l)

Location	Parameter	Count	Minimum	Maximum	Mean	Standard deviation	95% confidence levels
9	SS	38	8	48	23	11	23±3
	VSS	38	2	15	6	3	6±1
	BOD_5	31	2	16	6	4	6±1
	PO_4-P	38	0.4	8.8	2.5	1.5	2.5±0.5
	NH_3-N	37	2.0	16.0	6.9	3.6	6.9±1.1
10	SS	38	2	42	14	7	14±2
	VSS	38	1	11	5	2	5±1
	BOD_5	31	1	15	6	4	6±1
	PO_4-P	38	0.4	6.6	2.2	1.2	2.2±0.4
	NH_3-N	37	0.5	13.0	5.7	2.9	5.7±1.0
5*	SS	56	6	127	27	23	27±6
	VSS	56	1	44	9	7	9±2
	BOD_5	57	1	67	14	12	14±3
	PO_4-P	55	0.3	8.9	3.7	2.3	3.7±0.6
	NH_3-N	57	0.1	13.0	4.2	2.2	4.2±0.6
6*	SS	57	6	106	22	19	22±5
	VSS	57	1	30	8	6	8±2
	BOD_5	57	1	42	11	10	11±3
	PO_4-P	54	0.2	7.8	3.3	2.2	3.3±0.6
	NH_3-N	57	0.1	13.0	4.1	2.2	4.1±0.6

FIGURE 38

PROCESS EFFICIENCY
BOD CONCENTRATION
VS
TIME



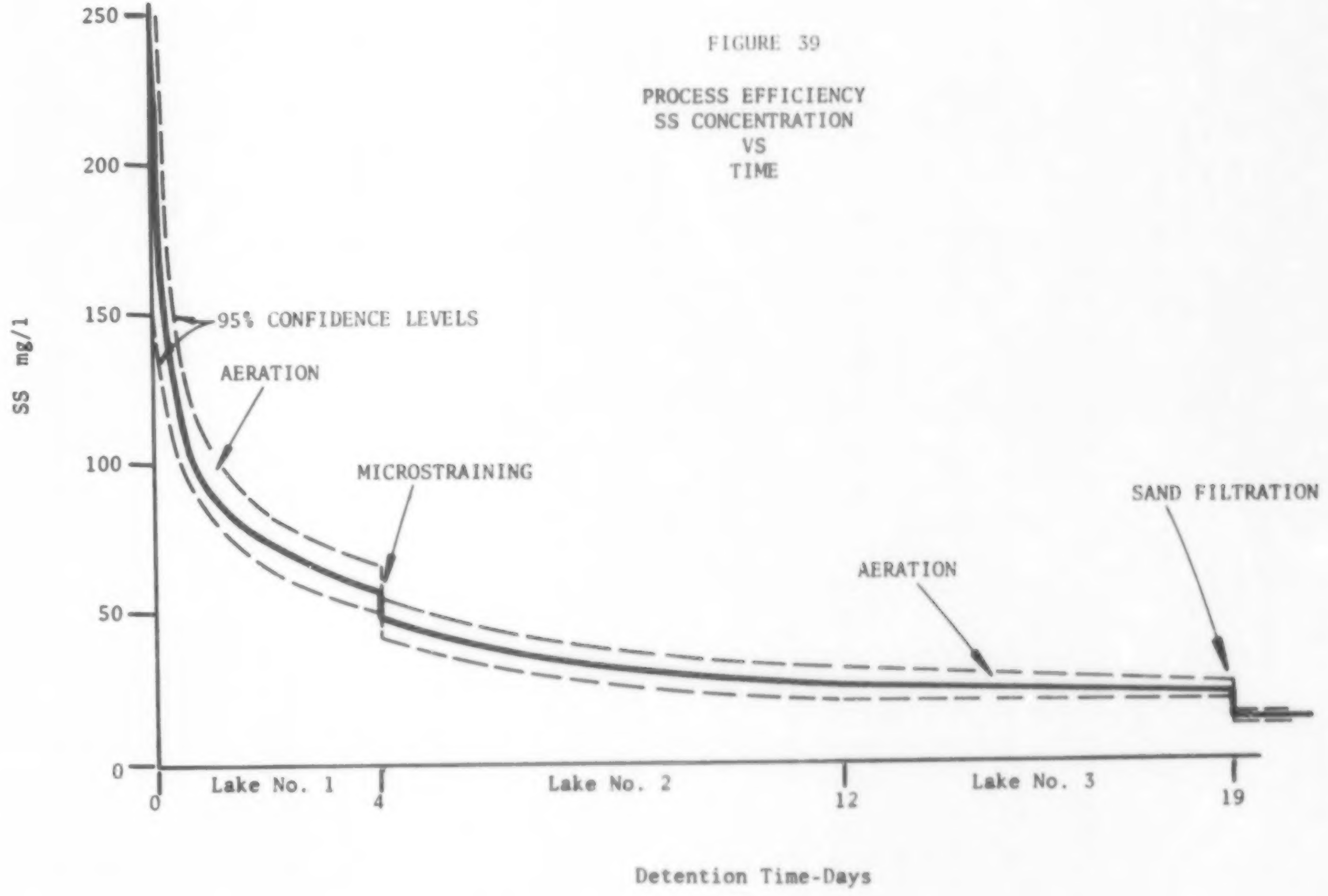


FIGURE 39

PROCESS EFFICIENCY
SS CONCENTRATION
VS
TIME

For the project service area of 212 acres, the total influent BOD_5 and SS loadings to the lakes system were calculated assuming the same amount of annual runoff volume (13,852,000 cu ft) during the pre-construction studies. The influent BOD_5 and SS values used were 75 mg/l and 239 mg/l respectively, observed during the post-construction studies. Total influent loadings, based on BOD_5 and SS values of 140 mg/l and 350 mg/l respectively observed during the pre-construction studies, are composed in Table 11

TABLE 11. ANNUAL BOD_5 AND SS REMOVAL WITH MOUNT CLEMENS CONCEPT

Parameter	Annual influent loadings[a] lbs/acre/yr	Annual effluent loadings[a] lbs/acre/yr	Removal percentage
BOD_5	570[b]	24	96[b]
	305[c]		92[c]
SS	1427[b]	57	96[b]
	974[c]		94[c]

[a] Annual influent and effluent loadings are based on the pre-construction period annual runoff volume

[b] Pre-construction period sampling data

[c] Post-construction period sampling data

The BOD_5 and SS loadings discharged to the Clinton River were calculated based on the same amount of annual runoff volume (13,852,000 cu ft). These loadings are shown in Table 11.

Table 11 indicates that there is a 92 to 96 percent reduction in BOD_5 and a 94 to 96 percent reduction in SS, depending upon the pre-construction or post-construction period overflow quality.

The Dissolved Oxygen (DO) concentrations in Lakelet No. 3 (Table B-1, Sampling Location 9) were consistently near the maximum value. The DO tests were performed on Sand Filter influent which originated from approximately one foot above the bottom of Lakelet No. 3. These results indicate that a favorable DO concentration was present throughout the lakelet.

The pH of the water in Lakelet No. 3 was always within 6.7 and 7.9, which is an acceptable range of values.

One major concern during the operation of the lakes system was the large quantity of algae experienced in Lakelet Nos. 2 and 3. The quantity was sufficient, at times, to produce a definite green tint to the water. Algae was expected in the lakes and was intended to be removed by the Sand Filter during the recirculation cycle. However, during the course of the study, it was found that the Sand Filter could not remove the algae. It is considered necessary that a more effective method for algae removal was required, or phosphorus removal be incorporated into the treatment process of any future facility prior to flow to the lakes system.

The Clinton River sampling data (Table C-1) indicated that the lakes system effluent with or without sand filtration was of a higher quality than the river. The average flow in the river is 468 cfs and the effluent flow was 1.5 cfs. Hence, the effluent could not improve the river quality by dilution. This would still be the case during a 7 day, augmented drought flow (10 year frequency) of 165 cfs. However, it should be noted that the Demonstration Facility did improve the river quality in that it removed 90 percent of the annual pollutional load on the river from combined sewer overflows originating from the service area of 212 acres.

At the end of the one year study period, an average 9 inches of sludge was observed on the bottom of Lakelet No. 1. This amounts to 28,144 cu ft or 133 cu ft/acre/yr. The Demonstration Facility was not intended to incorporate sludge removal. However, for a permanent facility, some method of sludge removal would be required.

During the fall of 1972 near record high water levels were being ap-aproached in the Great Lakes which caused a rise in water elevation in the Clinton River, which, in turn, created inflow problems in the city's combined sewer system. The improper functioning of the flap gates allowed the river to enter the system which resulted in the total flow exceeding the pumping capacity of the wastewater treatment plant. This situation caused a diluted wastewater to overflow into the Demonstration Project. This produced combined sewer overflow data (Table B-1) when no rainfall had occurred. The lakes system influent had BOD_5 and SS of 75 mg/l and 239 mg/l respectively, whereas the overflow quality during the pre-construction period had BOD_5 and SS of 140 mg/l and 350 mg/l. This situation has been compared in Table II while discussing the annual BOD_5 and SS removal with the Mount Clemens concept. It is our conclusion that the Mount Clemens concept can remove a minimum of 90 percent of the pollutional load from combined sewer areas.

Comparison with Water Quality Standards

As previously described, the development of the multi-purpose concept included the use of Lakelet No. 3 as a recreational facility involving fishing and boating and also the use of some of the process-water for lawn sprinkling. For these to be acceptable uses, the water quality of the Lakelet must conform to certain minimum standards.

The State of Michigan has listed general water quality standards for the following water uses:

A. WATER SUPPLY
  (1) Domestic
  (2) Industrial
B. RECREATION
  (1) Total Body Contact
  (2) Partial Body Contact
C. FISH, WILDLIFE AND OTHER AQUATIC LIFE
D. AGRICULTURAL
E. COMMERCIAL

The general standards are listed under the following parameters:

(1) COLIFORM GROUP
(2) DISSOLVED OXYGEN
(3) SUSPENDED, COLLOIDAL & SETTLEABLE MATERIAL
(4) RESIDUES
(5) TOXIC & DELETERIOUS SUBSTANCES
(6) TOTAL DISSOLVED SOLIDS
(7) NUTRIENTS
(8) TASTE & ODOR PRODUCING SUBSTANCES
(9) TEMPERATURE
(10) HYDROGEN ION
(11) RADIOACTIVE MATERIALS

The standards which were considered most important in relation to the multi-purpose use of Lakelet 3 are indicated in Table 12 along with the respective data obtained for Lakelet 3. Based upon these results, it is felt that the studies conducted under the Demonstration Project for the Mount Clemens Collection and Treatment facility have demonstrated the capability of this treatment concept to acceptably renovate combined sewer overflows for the multi-purpose uses as proposed. However, it should be noted that toxic and deleterious substances (which might affect the use of fish as food) were not studied, and it is recommended that additional studies be undertaken if the production of fish as food (as opposed to the sport of "catch and release") is a desired aspect of similar projects. It should also be noted that phosphorus removal appears necessary to meet fully the nutrient standard.

A monitoring program should be established. The combined sewer service areas should be checked for sources of toxic and deleterious substances. Control chemicals for the treatment system such as algacides should also be checked for such properties.

TABLE 12. SELECTED WATER QUALITY STANDARDS AND LAKELET 3 ATTRIBUTES

PARAMETER	MICHIGAN WATER QUALITY STANDARDS			LAKELET NO. 3 RESULTS
	PARTIAL BODY CONTACT (Fishing-Boating)	FISH, WILDLIFE (Growth & Propagation)	SPRAY IRRIGATION	
Fecal coliform Group (MPN/ 100 ml)	Not more than 1000	Not more than 1000	Not more than 1000	Less than 20
Dissolved oxygen (mg/1)	Average daily not less than 5 No single sample less than 4		Not less than 3	More than 6.0
SS (mg/1)	No objectionable turbidity, color or deposits	No objectionable turbidity, color or deposits	No objectionable turbidity, color or deposits [100]	Average of 23
Toxic & Deleserious substances	No injurious amounts	*	No injurious amounts	Not studied
Nutrients (mg/1)	Limit to prevent stimulation of algae, weeds & slimes	Limit to prevent stimulation of algae, weeds & slimes		PO_4 in terms of P=1 to 5 Ave. 2.2
Hydrogen Ion (pH)	6.5 to 8.8, but uniform within 0.5 unit	6.5 to 8.8, but uniform within 1.0 unit	Uniform within 0.5 unit	6.7 to 7.9

*Not to exceed 1/10 of the 96-hour median tolerance limit obtained from continuous flow bioassays where the dilution water and toxicant are continuously renewed except that other application factors may be used in specific cases when justified on the basis of available evidence and approved by the appropriate agency.

SECTION VII

COMPARISON WITH A SEPARATION PROGRAM FOR PROJECT AREA

Separation of combined sewage can be achieved (presently with State approval) by the construction of either new sanitary sewers to receive the dry weather flow using the existing combined sewers to receive the storm water flow, or by construction of new storm sewers to receive the storm flow using the existing combined sewers to receive only the dry weather flow. Several characteristics of any combined sewer system must be considered in the selection, not only of new sanitary sewer vs new storm sewers, but, in fact, in the very decision as to whether, or not, to separate at all. For instance, the usual extended age of combined sewer systems, typically constructed with joints which are currently unacceptable for use in sanitary sewers (conducive to high infiltration), would tend to discourage its use as a sanitary sewer. On the other hand, the lack of adequate capacity based on currently acceptable standards for use as storm sewers might tend to discourage the use of the combined sewer as a storm sewer.

Moreover, there are certain economic aspects which must be weighed in the overall decision about "separation". A location in a "downtown" congested area, sometimes with narrow streets having two or three sewers already in existence, can substantially increase construction costs. Furthermore, significant inconvenience to the public at large by virtue of interference with traffic, is sometimes considered intolerable.

It is also worthy of noticing at this time that total "separation" may not achieve the desired goal of adequately reducing the pollutional impact on the receiving stream.

> "Runoff from street surfaces is generally highly contaminated. In fact, it is similar in many respects to sanitary sewage. Calculations based on a hypothetical but typical U.S. city indicate that the runoff from the first hour of a moderate-to-heavy storm (brief peaks to at least 1/2 inch per hour) would contribute considerably more pollutional load than would the same city's sanitary sewage during the same period of time. . ."[10]

Separation By Sanitary Sewer Construction

"Separation" by construction of sanitary sewers would involve the installation of a new sanitary sewer below the elevation of the existing combined sewer in order to allow the reconnection of existing building service connections to the new sewer. The new sewer must generally be located near the old sewer but care must be taken not to disturb the existing sewer during construction because the existing sewer must be saved for use as a storm drain. The new sanitary sewer must, of course, then be extended to the nearest available sanitary sewer outlet. The existing sewer (now a storm sewer) must be sealed off from all sanitary sewer connections and perhaps an investigation may be necessary (by means of a closed circuit television device) to make sure that this is accomplished. Any existing overflow devices must be modified to allow the flow of the storm water to discharge directly to the receiving stream. This type of separation program may yet contain some bad features. For instance, some rain water downspouts may be connected to the building service connections which would have a much more adverse affect on a sanitary sewer system than it did when it was a combined sewer system. Foundation drains connected to the building service connections may also have a similar undesirable affect. To achieve the disconnection of all sources of storm water and/or ground water would be an enormous, if not impossible, task.

The basis for the estimate of construction cost for the installation of a new sanitary sewer system in the demonstration project service area is presented in Table 13.

The total cost of separation by the construction of new sanitary sewers was not developed, however, since it was determined that the average cost per lineal foot was almost double the cost of separation achieved by the installation of new storm sewers, the discussion of which follows immediately.

Separation By Storm Sewer Construction

"Separation" by means of the construction of new storm sewers was found to be a much more economical method (not counting the effect of infiltration) than by the installation of new sanitary sewers. As mentioned above, upon the installation of the new storm sewer, the existing combined sewer would continue to be used for dry weather flow, therefore, it is imperative to eliminate the introduction of all rain water or ground water into the existing system.

Among the problems accompanying this type of solution to the separation problem is the fact that the existing combined sewer (now a sanitary sewer) is grossly oversized for the sanitary sewage flow resulting in extremely low velocities which will cause excessive settling of suspended solids material in the sewers. On the other hand, construction of new storm sewers may be very desirable in some areas where additional capacity is necessary to alleviate possible local flooding conditions.

The cost of construction of new storm sewers is less than that of sanitary sewers primarily because of two items. (1) The storm sewers can usually be constructed at shallower depths since it is not necessary to intercept building service connections. (2) The disconnection and reconnection of building service leads to a new sanitary sewer is a very substantial cost item.

The basis for cost estimate for separation by the installation of storm sewers is indicated in Table 14. On this basis, a construction cost estimate for a separation program for each street in the project service area was developed. As in the development of total project cost for all

TABLE 13. BASIS FOR COST ESTIMATE FOR SEPARATION BY INSTALLATION OF SANITARY SEWERS

Item	Cost per lineal foot[a]	
	In concrete pavement streets	In bituminous pavement streets
Pipe material	$ 2.80	$ 2.80
Bedding	0.60	0.60
Sand backfill	10.00	10.00
Excavation (labor-equipment)	6.60	6.60
Pavement removal & replacement	16.00	7.50
Manholes	2.50	2.50
Lead connections	21.50	20.00
Sub-total	$60.00	$50.00
30%[b]	18.00	15.00
Total project unit cost	$78.00	$65.00

[a] Based on 1973 construction; ENR Construction Cost Index of 1800

[b] Allowance for contingencies, engineering, inspection, legal, bonding, administration, and capitalized interest.

other construction, there was included a 30% allowance on top of construction costs to cover items such as contingencies, engineering, inspection, legal, bonding, administration, and capitalized interest. The resultant average cost per acre in the two project service areas amounted to approximately $12,000 per acre. Please note, however, that this service area is predominately residential in nature and it is estimated that the total project cost in an urban area containing a "downtown" or central business district would be approximately $15,000 per acre.

Economic Comparison of Separation vs Collection and Treatment

The total cost for separation in the project service area was estimated to be $2,544,000. For comparison purposes, it was assumed that such a separation program would have a life of about thirty years before it might be considered outmoded. Moreover, a project of this nature would most likely be funded by means of a bond issue which would be repaid over the same thirty year time span and having a 6% interest rate. These considerations result in an annual cost of approximately $185,000.

TABLE 14. BASIS FOR COST ESTIMATE FOR SEPARATION BY INSTALLATION OF STORM SEWERS

Sewer size, in.	Cost per lineal foot[a]		
	Concrete pavement	Bituminous pavement	Tunnel construction
12	$19.50	$15.50	
15	20.00	16.00	
18	20.50	16.50	
21	32.50	23.00	
24	33.00	24.00	
30	36.00	26.50	
36	44.50	35.00	
42	50.00	43.00	
48	55.00	48.50	$110.00
54			125.00
60			150.00
66			175.00
72			200.00
78			210.00
84			225.00

[a] Based on 1973 construction; ENR Construction Cost Index of 1800

The total cost for the collection and treatment facility (for combined sewer overflows) for this project service area could not be precisely evaluated. The extenuating circumstances causing this are twofold:

1. Operation and maintenance costs of the facility may be extremely variable over the 30-year comparison period
2. The entire 24 acre site was purchased for use as a park site as well as for use as part of the treatment process.

However, for purposes of comparison, we have assumed a constant operation and maintenance cost equal to the current annual cost of $45,000. Moreover, it was assumed that only about 10 acres of the total site was necessary as a portion of the treatment facility. Furthermore, the cost of the 10 acre site was included as part of the capital cost even though land would not normally be considered "lost" and, in fact, may appreciate in value over the 30 year time period rather than depreciate.

It was assumed that the cost of the 10 acre treatment facility site was $50,000. While this cost may appear low, the Mount Clemens site primarily consisted of a sanitary landfill which substantially increased the construction cost of the treatment facility.

The total project cost for construction of the demonstration project was $758,000. Assuming the same 30 year life and bond issue, the annual cost would be $55,000. The annual cost of operation and maintenance was $45,000 resulting in a total annual cost of approximately $100,000.

Thus, the comparison between the annual cost of a separation program and the annual cost of a collection and treatment facility shows that there would be an approximate savings of $85,000 per year by the selection of the collection and treatment facility alternative.

Storm Water Pollution

If a combined sewer system is separated by constructing new storm sewers, these storm sewers would convey the surface runoff to the receiving stream.

Assuming the pre-construction period annual runoff volume of 13,852,000 cu ft, the annual runoff would contain 196,200 lbs of SS and 14,700 lbs of BOD_5. These values can be expressed as 925 lbs/acre/year or 30 lbs/acre-inch of rainfall for SS and as 70 lbs/acre/year or 2 lbs/acre-inch of rainfall for BOD_5. These loadings are based on storm water quality having an average SS of 227 mg/l and BOD_5 of 17 mg/l.[11]

SECTION VIII

OPEN SPACE, RECREATION, AND TRANSITIONAL LAND USE

No money was available for construction of the park facilities during the study of the concept, hence, the actual use of the park facilities and the accompanying study of the possible merits of spray irrigation were impossible to evaluate within the time available for completion of this study. Therefore, a literature review was conducted to produce information to be correlated with lakelet sampling data for evaluating the potential for future use of the water orientated recreational aspects of the facility.

> "Water is a focal point of outdoor recreation. Most people seeking outdoor recreation want water--to sit by, to swim and to fish in, to ski across, to dive under and to run their boats over. Swimming is now one of the most popular outdoor activities and is likely to be the most popular of all by the turn of the century. Boating and fishing are among the top ten activities. Camping, picnicking, and hiking, also high on the list, are more attractive near water sites."[12]

As our recreational lands and waters become more difficult to acquire, it becomes ever more important to identify possible new areas, especially in a congested urbanized area.

This section of the report deals with the multi-purpose benefits potentially achievable from a Combined Sewer Overflow Collection and Treatment Facility. Although several aspects of this multi-purpose facility were not able to be developed and tested during the course of the project, the discussion presented herein attempts to communicate what is envisioned as the concept's development potential in the provision of open space and recreation, and as an acceptable transitional land use area.

The reason for providing open space and recreational land areas are manifold. The percentage of participants engaging in specific activities will usually increase as increased opportunities produce increased demand. A continuing enlargement of this percentage will be experienced because of the increase in leisure time which is expected in the future.

The insecurity and tension of city life can bring city dwellers to a state of anxiety or depression from which they seek escape. For these and other reasons the city dweller frequently turns to forms of recreation that give emotional release from whatever in his life situation he finds disagreeable. In addition to the value which recreation has for the health and contentment of families, there is evidence in support of the contention that recreational and open space areas, properly located, planned, and maintained, have a beneficial effect on land and real estate values.

To further list the multitude of reasons for incorporating open space, recreation, and transitional land areas within an urban environment would only repeat what has been often said by many authorities.

It is apparent that American communities are significantly deficient in these areas due to ill-planned land uses. Presently, either excessive land values render it too expensive to allot adequate open space and recreational land areas or existing open space is consumed by expanding industrial or commercial enterprises.

While keeping all of the above mentioned attitudes in mind, the Treatment-Park Concept was thus developed for the purpose of achieving the greatest benefits from a publicly funded project.

Figure 2, again shows the proposed development plan for the treatment-park site. The recreational activities proposed during the summer months consist of canoeing, small boat sailing, fishing, picnicking, and scenic walks. During the winter months, ice skating would be the most probable activity.

These types of activities would hopefully result in a family oriented park. Vigorous physical activities (basketball, football, etc.) were not contemplated (though not discouraged) since the percentage of the total public who would become involved in a strenuous type of recreational activity would be small. However, as indicated in Figure 40, when the sewage treatment plant personnel and the consulting engineering staff got together on a brisk winter day, an enjoyable and invigorating afternoon on Lakelet No. 3 was had by all.

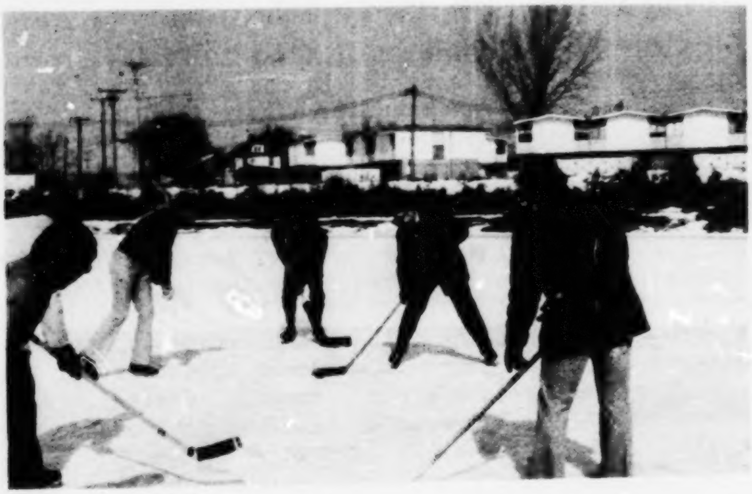


FIGURE 40. LAKELET NO. 3 RECREATION

The treatment-park site was constructed adjacent to the existing sewage treatment plant as previously discussed. In addition to the aforementioned reasons for so doing, was the fact that the location was ideally situated to allow development of a park site which would serve as a buffer zone between imminent commercial developments and existing residential areas. As seen on the general zoning map in Figure 41, the area immediately north of the site is an existing residential area and the land immediately south is zoned commercial.

The facility's location also enhances the recreational area distribution within the City. This can also be seen in Figure 41. It should be noted that the area zoned open space in the northwest corner of the City also incorporates a county complex and other public facilities and is not actually set aside for an outdoor recreational area. "Shadyside Park," located next to the spillway on the southeast side of the City, is the only other large park facility within the City which is not in some way associated with a school facility. Therefore, the location of the treatment-park site, with respect to the rest of the City, tended to increase its value as a park site.

Table 15 indicates typical values for planning recreational developments for various types of uses. A commonly recognized standard for recreational open space in urban areas is one acre for every hundred persons. The City of Mount Clemens has set this standard as its goal for open space in its general development plan dated June 20, 1970. However, the one acre for every hundred persons standard will be extremely difficult for the City to achieve since, as indicated in Table 16, the City's ultimate present capability for park and recreation development (without additional purchase or condemnation) is only 155 acres. Thus, the programed development provides only 0.78 acres per one hundred persons, including the treatment-park site. This reaffirms the importance of utilizing the Treatment-Park Concept as an effective means for supplying additional open space and recreation.

Figures 42 and 43 show the project site during the post-construction study period. Figure 42 shows the treatment park with the central business district in the background. The open space immediately south of the site is awaiting a development and the area west of the site is anticipated to accept a new high rise building. Figure 43 shows the site and the land usage looking east including the race track, expressway, and a commercial area in the adjoining township. Figure 43 also shows the open area south of the river where a new retention basin for combined sewer overflows is proposed as discussed elsewhere in this report.

A study of Figures 42 and 43 would indicate the favorable location of the treatment-park site as a public facility development for the purposes of supplying open space, recreation, and transitional land use areas.

The cost analysis for a multi-purpose facility of this type is extremely difficult due to certain overlapping and intangible items inherent in the facilities. For example, if open space were the only consideration for land to be provided between two subdivisions, recreation would surely become an incidental benefit.

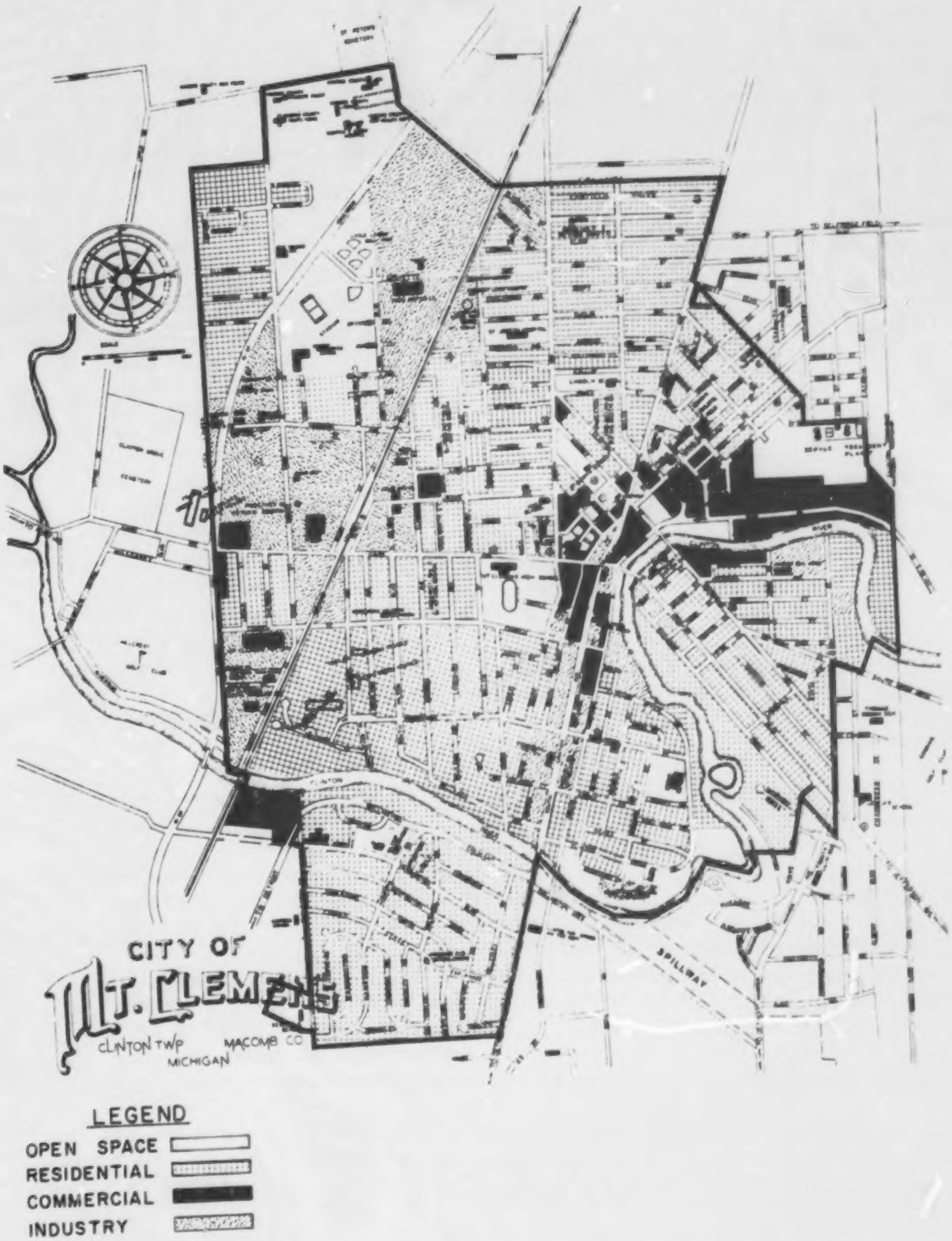

CITY OF
MT. CLEMENS
CLINTON TWP
MICHIGAN
MACOMB CO
SPILLWAY
LEGEND
OPEN SPACE
RESIDENTIAL
COMMERCIAL
INDUSTRY

FIGURE 41. GENERAL ZONING MAP

TABLE 15. RECREATION STANDARDS FROM SELECTED AREAS
(Acres per 1,000 population)

Source	Local	County	Regional	State and Open Lands	Total
Comprehensive Plan for Wisconsin, Outdoor Rec., 1966	10	15			25
Baltimore Regional Planning Council	4.5		15	25	44.5
Northeast Illinois Planning Commission, 1962	10	15	15		40
Lake County, Illinois 1966	10	10	10		30
National Association County Officials, 1967	10	10	10		30
Detroit Regional Planning Commission	10	15	15	15	40
Milwaukee County Park System	10	15	10	80	115
Regional Plan Association of New York	12 acres or 5% of Co.				
Planning Commission of Lackawana, 1963	7	15		45	67
Connecticut Dept. of Agriculture and Natural Resources, 1966	10		10	10	30
New Jersey Dept. Conservation and Economic Development	5	9		20	34
National Recreation and Park Association, 1965	10		15	65	90
Genesee County, Michigan		20			20

TABLE 16. CITY OF MOUNT CLEMENS PROGRAMED PARKS AND RECREATION FACILITIES

Facility	Acreage
Existing park sites	86.7
Playgrounds as part of school facilities	33.1
Proposed new facilities (including treatment-park site)	35.5
Total	155.2

The initial cost for such facilities as these would be, of course, the actual purchase of the land. This cost will be dictated by local land values which would vary depending on the competition, that is, whether the land would have become industrial, commercial, or residential areas. Further costs would include development, operation, and maintenance costs and a long term cost could be the loss of tax revenue from the land.

A benefit to cost analysis would entail estimating an individual dollar value for each portion of the facility. In addition to the open space, recreation, and transition land use portions, some of the more intangible benefits should be included in the evaluation such as a possible increase in local land values. Then, each portion must be evaluated concerning its individual significance to the particular project as a whole.

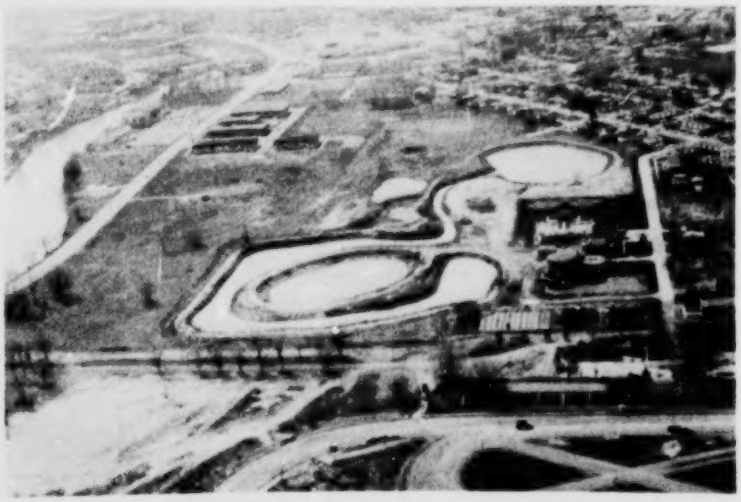


FIGURE 42. AERIAL VIEW OF SITE VIEWING WEST

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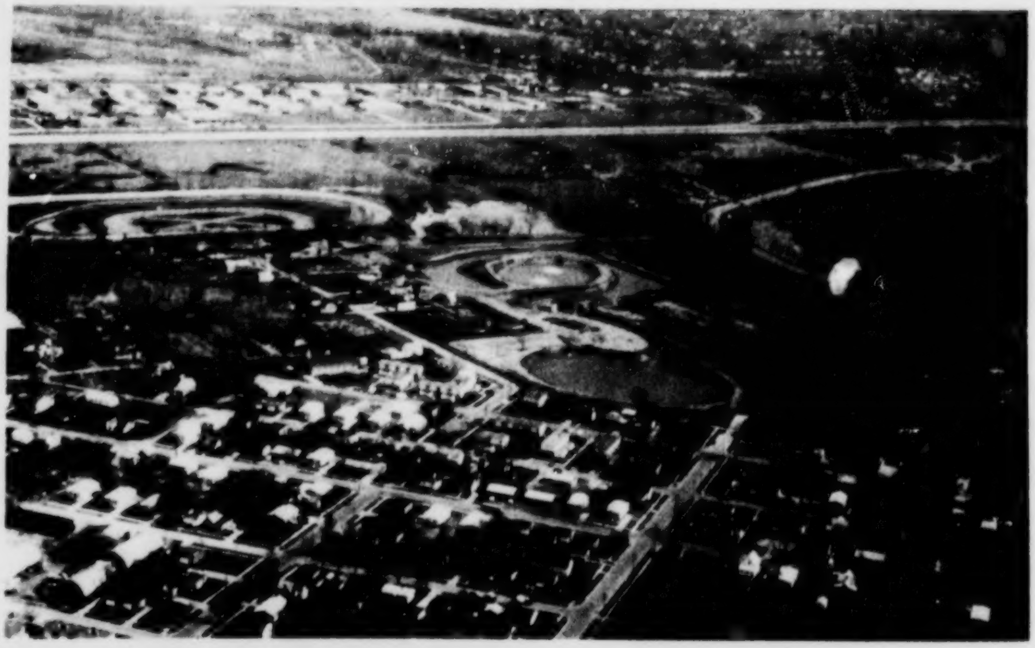


FIGURE 43. AERIAL VIEW OF SITE VIEWING EAST

SECTION IX

EPILOGUE-GENERAL DISCUSSION ON THE MOUNT CLEMENS CONCEPT

The City of Mount Clemens was, as previously mentioned, originally solicited as the sponsor for the demonstration of the multi-purpose concept for use in solving its combined sewer overflow problem. Hence, with the successful demonstration of the concept, the City has undertaken the development of a Wastewater Collection System and Multi-Purpose Treatment Facility project aimed at "reducing the frequency, magnitude, and polluting content of overflows of combined sewage, industrial wastes, and storm water from the City's sewer system to the Clinton River", as required in the Stipulation between the Water Resources Commission and the City of Mount Clemens dated November 16, 1967.

It is therefore the purpose of this section to present the city-wide project as it was proposed to the City and to also suggest general design procedures for universal application of this concept. These general design procedures were the product of the demonstration project studies and of the experience gained in designing the major City facility, including the ensuing work in obtaining approvals for a State construction permit and a Federal Construction Grant.

CITY WIDE PROJECT FOR MOUNT CLEMENS

It was recommended that a substantial portion of the City (1500 acres out of a total of 2100 acres) remain "unseparated" and that a combined sewage interceptor be installed to collect the overflows and convey them to a retention basin, the contents of which would be withdrawn at a slow uniform rate for further treatment. For the remaining portions of the City (600 acres out of a total of 2100 acres) it was recommended that "Separation", by constructing new collecting sanitary and/or storm sewers, would better serve the integrity of a total waste treatment works system. This Plan is outlined in Figure 44, General City Wide Collection System Plan.

The City of Mount Clemens was divided into 5 areas described as follows:

Area 1 - From Northern City limits to the Clinton River, and from the treatment plant to Grand Trunk Railroad.

Area 2 - Southeast portion of the City.

Area 3 - Southwest portion of the City, south of the Clinton River and the area north of the river from South Wilson Blvd. to Barbara St.

Area 4 - Area around West Breitmeyer Place.

Area 5 - From Northern City limits to the Clinton River and from Grand Trunk Railroad to Western City limits.

A cost analysis has indicated that:

(i) Areas 1, 2, and 3 should remain "unseparated" except for the small northern most portion of Area 1, where new storm and sanitary sewers are recommended, and that a better alternative is to collect and treat the combined sewer overflows.

(ii) Areas 4 and 5 should be separated; Area 4 by constructing storm sewers, and Area 5 by constructing sanitary sewers and some storm sewers.

A summary of this analysis is shown in Table 17.

TABLE 17. COST ANALYSIS; SEPARATION VS COLLECTION & TREATMENT

Priority Area	Acres	Cost/acre separation ($)	Cost/acre collection & treatment ($±)	Minimum cost for area ($)
1	1002	15,000	9,260	9,276,000
2	179	15,000	4,290	768,000
3	290	15,000	6,740	1,956,000
4	53	10,000	10,750	560,000
5	565	2,500	3,750	1,440,000
TOTAL	2089*	-	-	14,000,000

*This area does not include portions of the city already separated.

ENR CONSTRUCTION COST INDEX: 1850

Operational Description

The collection and treatment scheme for the city-wide project is indicated on the Flow Diagram shown on Figure 45. The design basis is indicated in Table 18 and discussed separately immediately following this subsection. The project involves the interception of overflows (5-year storm) from combined sewers at various locations throughout the City and conveying them by gravity to the wet well of the combined sewer overflow main pump station at the retention basin site. The overflows will be lifted into a discharge well of the station after which they will flow through the sedimentation-resuspension chambers (hereafter called the SRC). The overflows will then discharge into an aerated retention basin.

The SRC consists of three individual sedimentation tanks in series through which the wastewater is conveyed prior to discharging into the retention basin. The SRC is designed to provide settling of a substantial portion of the suspended material during the inflow period. Upon cessation of any inflow, the settled material is to be resuspended by means of a series of dual ejector units which utilizes forced air and high pressure water jets to create a turbulent mixing effect. The contents of the SRC are then conveyed to the treatment facility.

The retention basin will receive the overflows and provide aeration from floating aerators. At times when the retention basin capacity is exceeded (5-year storm), the excess will overflow into the chlorination basin thence discharge to the Clinton River. The retention facility provides a minimum of 3 hours detention while providing sedimentation, aeration, and disinfection. It is expected that the settled, aerated, and chlorinated emergency overflows from the retention basin will have no adverse impact on the Clinton River.

The wastewater is to be withdrawn from the retention basin site at a controlled 4 mgd rate and conveyed to the existing Demonstration Project site for treatment. The selection of the 4 mgd rate is discussed under the subsection 'Design Basis'.

A flushing system has been provided for cleaning the probable accumulation of sludge in the bottom of the retention basin and the SRC. The system incorporates jet flushing nozzles located around the perimeter of the retention basin, the dual ejector units for the SRC, plus fire hose connections to the flushing system for SRC and discharge well cleaning.

The City of Mount Clemens has elected to adopt the Macomb County-Detroit Metro Water Department (DMWD) Regional System as its method of dry-weather-flow sanitary sewage disposal, and the City intends to connect to this system when it is made available to the City. The DMWD Interceptor is expected to be completed in the early summer of 1976.

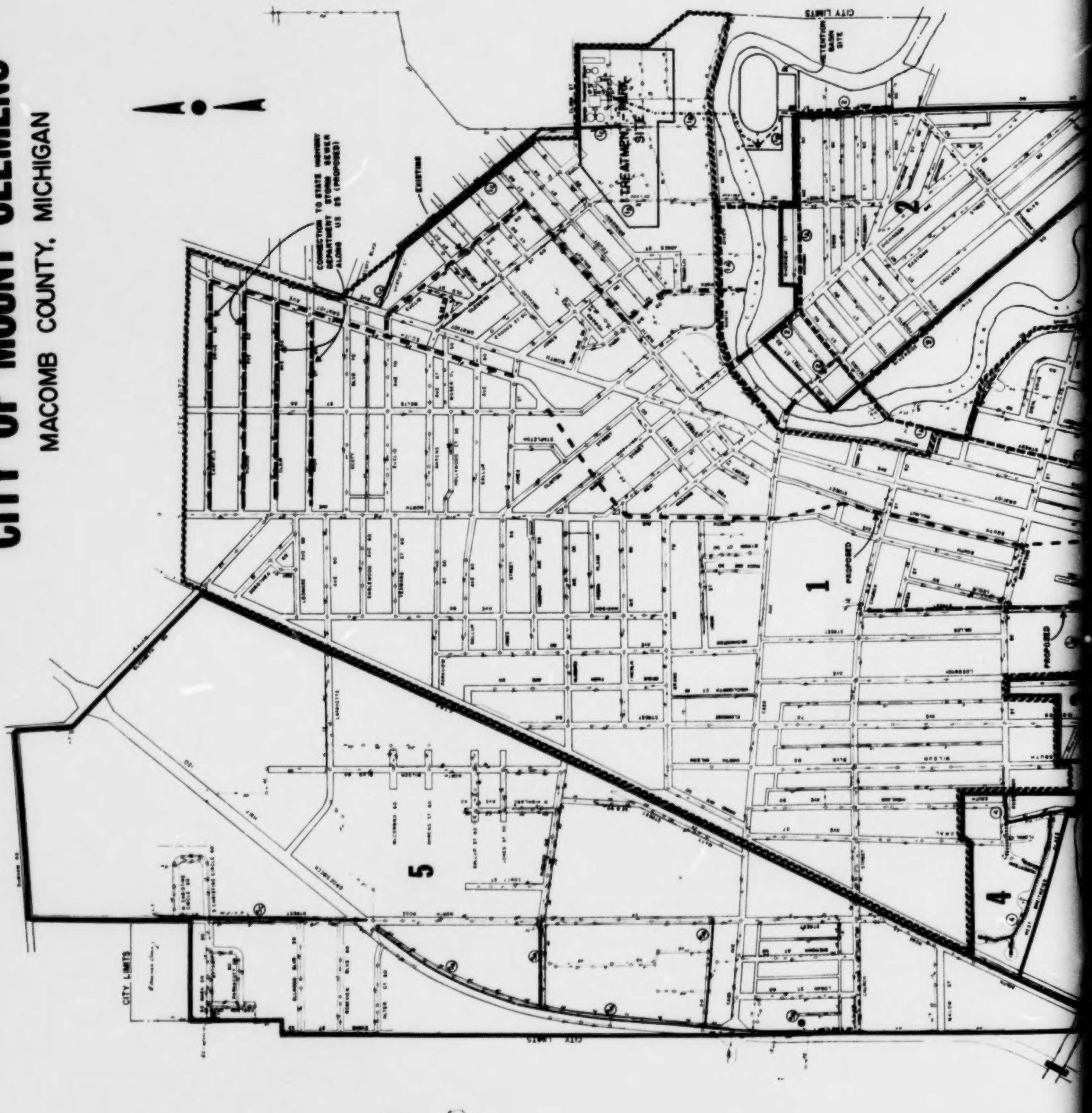
Therefore, since the existing wastewater treatment plant will be phased out of service, both the primary and secondary settling tanks will be modified to receive flows from the retention basin site. These units are to be used to remove the settled material which will have been captured and resuspended by the SRC. Moreover, future chemical additions for phosphate removal will occur at this location. The sludge from the settling tanks will be disposed of by means of the regional DMWD sanitary sewage disposal system.

The existing chlorination chamber will be utilized to insure that no pathogenic organisms are discharged into the lakelet system.

High rate pressure sand filtration was proposed as the next unit process for the purpose of polishing the chemically treated wastewater prior to discharge into Lakelet 1. It was felt that this unit process would have produced an extremely high quality water for flow into Lakelet 1.

CITY OF MOUNT CLEMENS

MACOMB COUNTY, MICHIGAN


CONNECTION TO STATE HIGHWAY DEPARTMENT STORM SEWER ALONG US 25 (PROPOSED)
EXISTING
TREATMENT PLANT SITE
RETENTION BASIN SITE
CITY LIMITS
PROPOSED
1
2
4
5

A

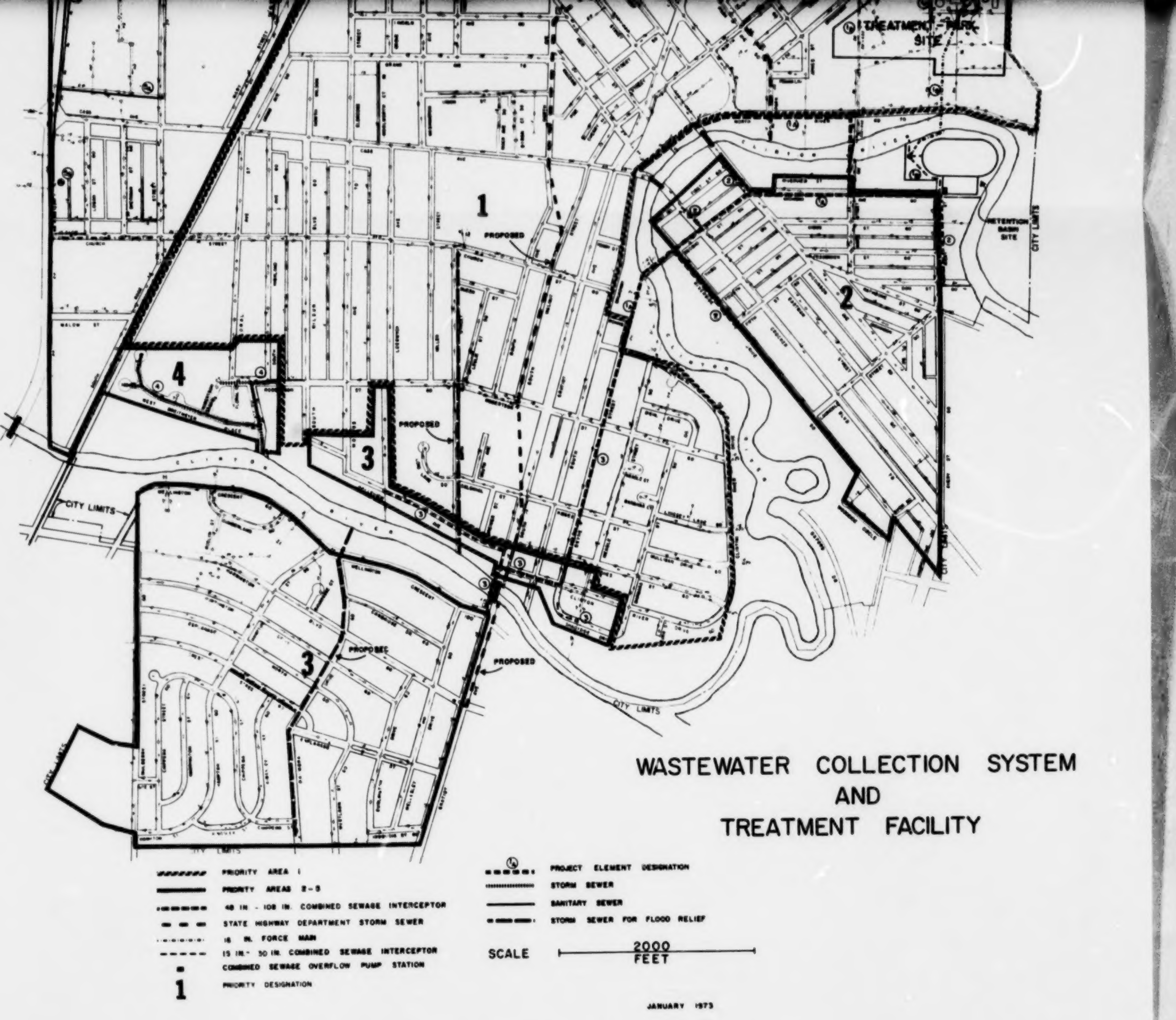
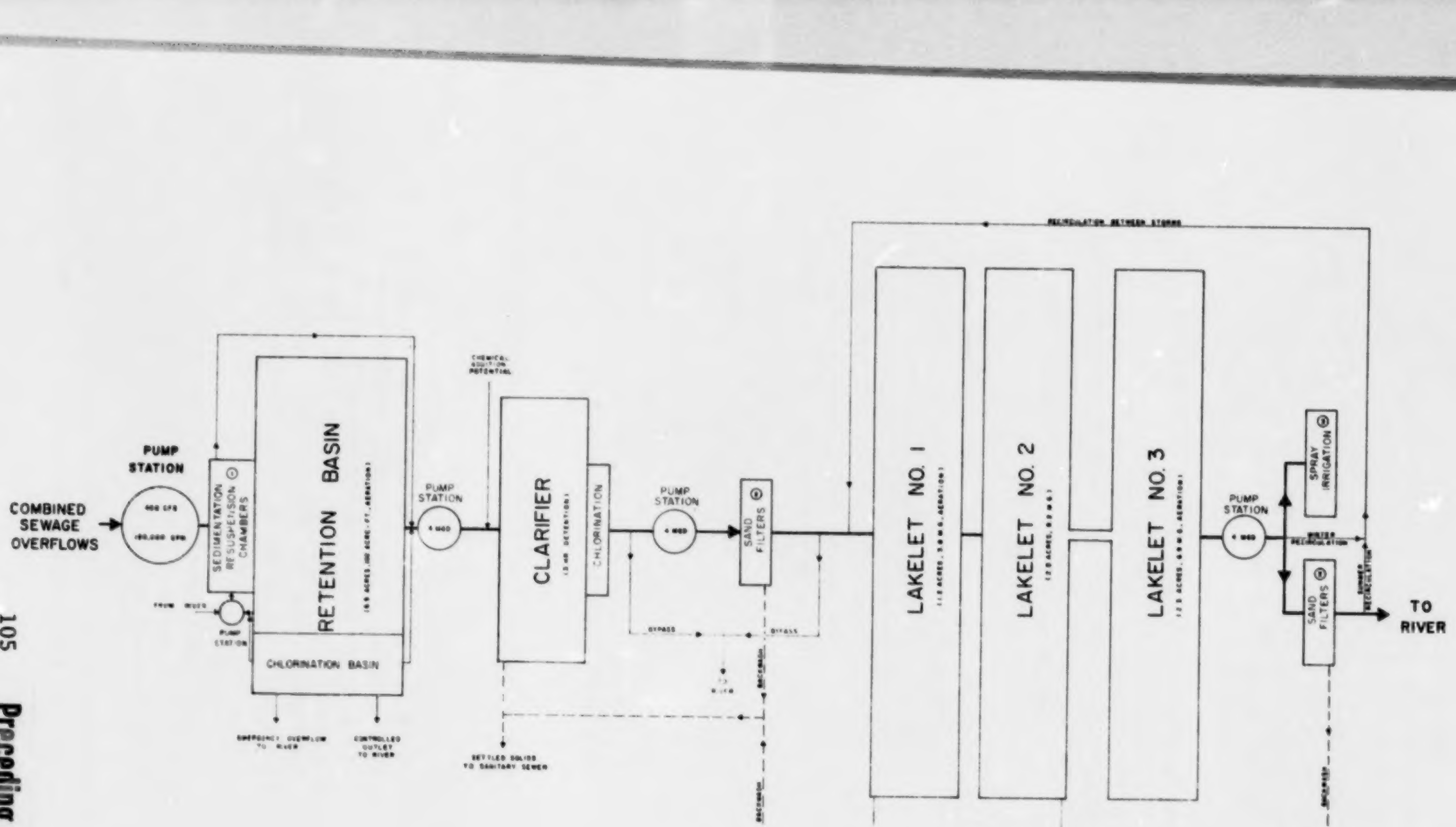

TREATMENT PARK SITE
RETENTION BASIN SITE
CITY LIMITS
PROPOSED
1
2
3
4
PROPOSED
PROPOSED
PROPOSED
CITY LIMITS
CITY LIMITS
WASTEWATER COLLECTION SYSTEM
AND
TREATMENT FACILITY
PRIORITY AREA 1
PRIORITY AREAS 2 - 3
48 IN. - 108 IN. COMBINED SEWAGE INTERCEPTOR
STATE HIGHWAY DEPARTMENT STORM SEWER
16 IN. FORCE MAIN
15 IN. - 30 IN. COMBINED SEWAGE INTERCEPTOR
COMBINED SEWAGE OVERFLOW PUMP STATION
1 PRIORITY DESIGNATION
PROJECT ELEMENT DESIGNATION
STORM SEWER
SANITARY SEWER
STORM SEWER FOR FLOOD RELIEF
SCALE
2000 FEET
JANUARY 1973

FIGURE 44. GENERAL CITY-WIDE COLLECTION SYSTEM PLAN


COMBINED SEWAGE OVERFLOWS
PUMP STATION
400 CFS
180,000 GPM
SEDIMENTATION RESUSPENSION CHAMBERS ①
FROM RIVER
PUMP STATION
RETENTION BASIN
(6.9 ACRES, 100 ACRE-FT., AERATION)
CHLORINATION BASIN
EMERGENCY OVERFLOW TO RIVER
CONTROLLED OUTLET TO RIVER
PUMP STATION
4 MGD
CHEMICAL ADDITION POTENTIAL
CLARIFIER
(3 HR DETENTION)
CHLORINATION
SETTLED SOLIDS TO SANITARY SEWER
PUMP STATION
4 MGD
SAND FILTERS ②
BYPASS
BYPASS
TO RIVER
BACKWASH
BACKWASH
RECIRCULATION BETWEEN STORMS
LAKELET NO. 1
LAKELET NO. 2
LAKELET NO. 3
LAKELET DRAIN TO RIVER
OVERFLOW TO RIVER
PUMP STATION
4 MGD
SPRAY IRRIGATION ③
WINTER RECIRCULATION
SAND FILTERS ②
SUMMER RECIRCULATION
BACKWASH
TO RIVER

NOTES

① SEDIMENTATION RESUSPENSION CHAMBERS AT 50% OF PEAK RATE HAVE 1 HR. DETENTION

② FILTRATION RATE 15 GPM/SQ FT

③ SPRAY IRRIGATION 14.3 ACRES

FLOW DIAGRAM

RETENTION AND TREATMENT FACILITIES

FIGURE 45

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However, due to a cut-back in the availability of Federal-State Grant funds, this unit was deleted from the presently proposed project. The deleted unit can be added at a later date if required.

The water will then enter Lakelet 1 which is to be converted from an aerated receiving retention basin to an aerated "flow-through" treatment lakelet. The water will then continue through the remaining lakelet system by gravity and is to be filtered through high-rate pressure sand filters prior to discharge to the Clinton River.

The Retention Basin (including the SRC and chlorination basin) has a 32 million gallon storage capacity. The 4 mgd pumping rate to the treatment facility will therefore dewater the retention basin (when full) in 8 days (thereby supplying an average of 4 day detention time). The existing clarifier at the sewage treatment plant will provide adequate solids removal at the 4 mgd rate and the lake system will supply about 5 days detention time. The Treatment-Park Site has been designated by the City for development as a Recreational Facility. Continued sampling of Lakelet 3 water quality will be performed to monitor its acceptability for recreational use and potential use for watering of the park greenery. Also, an application will be processed through the State of Michigan, Bureau of Sport Fisheries and Wildlife for the purpose of obtaining fish (Bass and Bluegill) to stock Lakelet 3. The growth and reproduction of these fish should be further studied to indicate the adaptability of fishing and fish production to the total concept.

Design Basis

The design basis for the city-wide project evolved mainly from the experience gained during the Demonstration Project studies, and is summarized in Table 18 for easy reference. The following discussion on the various wastewater facilities or process units, wherever necessary, is intended to explain the authors' views.

Existing Combined Sewers and the proposed Combined Sewer Overflow Interceptor

Our studies have shown that the existing combined sewers have the capacity for a present day 2-year storm. The proposed interceptor is designed for a 5-year storm using the Rational Method. Due to the 2-year capacity of the existing sewers, ponding occurs during storms exceeding the 2-year storm. The City has a storm sewer construction program for flood relief in the combined sewer service area. The relief sewers would discharge into the proposed combined sewer overflow interceptor. Thus, ultimately, the combined sewer system would be able to handle the 5-year storm.

The tunnel portion of the proposed combined sewer overflow interceptor is designed at a constant grade of 0.02 percent. Surcharge is assumed to provide 400 cfs capacity at 0.55 percent hydraulic gradient slope.

Each of the 19 overflow structures to the Clinton River would be maintained for emergency use in case of the combined sewer overflow main

TABLE 18. DESIGN BASIS - MOUNT CLEMENS CITY-WIDE PROJECT

Item no.	Wastewater facility or process unit	Design basis	Remarks
1	Existing combined sewers and interceptors	±2-year storm	Service area ±1400 acres
2	Combined interceptor	Rational method, Q=CIA. 'Detroit Curve' for 5-year storm $I = \frac{129.9}{t_c + 21.2}$	9 ft tunnel; designed to receive additional flows generated by the City's proposed sewer relief program
3	Main pump station	Peak flow 428 cfs.	5 pumps @ 35,000 gpm, 38,000 gpm with surcharge
4	Sedimentation-Resuspension chambers (SRC)	Peak settling velocity 0.0058 ft/sec	64 ft x 1074 ft x 10.5 ft deep (3 bay); minm. detention time 30 min
5	SRC resuspension	Ejector units - forced air and water	20 units per bay
6	Retention basin	Storage developed for mass diagram study of 10 years of rainfall data; 3 overflows from basin per year	102 acre-ft; 5-year storm would exceed retention capacity
7	Retention basin-aeration	170% of required aeration capacity due to restricted zone of influence	5-20 h.p. floating aerators, 2.32 lbs O_2/hp-hr
8	Chlorination basin	20 min detention time @ 200 mgd	2.8 mgal capacity

TABLE 18 (continued). DESIGN BASIS - MOUNT CLEMENS CITY-WIDE PROJECT

Item no.	Wastewater facility or process unit	Design basis	Remarks
9	Chlorinator (for emergency overflows to river)	5 mg/l dosage rate	8000 lbs/day capacity; liquid feed system; automatic additive rate system in conjunction with the 5 main pumps
10	Flushing system	Hydrants around SRC; fire nozzles around retention basin	Front end loader required for retention basin
11	Retention basin and SRC-dewatering	4 mgd rate - maxm. 5 mgd	2 pumps @ 2100-3500 gpm controllable rate
12	Grit chamber (existing)	1 fps	Future point of addition for chemicals
13	Clarifier	3 hrs detention time @ 4 mgd; provide flocculation	Modify existing primary and secondary clarifiers to flow in parallel
14	Sludge	Discharge to existing sanitary sewer and ultimately to the regional system	Monitor sludge density and volume
15	Chlorination at the	Chlorine or chlorine-dioxide; 15 min detention time @ peak hourly rate	Utilize existing chlorination basin and injection system
16	Sand filtration	15 gpm/ft^2	4-1.0 mgd pressure filters in parallel, backwash automatically one at a time

TABLE 18 (continued). DESIGN BASIS - MOUNT CLEMENS CITY-WIDE PROJECT

Item no.	Wastewater facility or process unit	Design basis	Remarks
17	Lakelet No. 1	1.2 acres with 3.8 mgal capacity	Provide 0.95 days detention at 4 mgd rate; mechanical aeration is proposed to reduce BOD_5 from 50 mg/l to 10 mg/l
18	Lakelet No. 2	2.8 acres with 8.2 mgal capacity	Provide 2.05 day detention time at 4 mgd rate
19	Lakelet No. 3	2.3 acres with 6.9 mgal capacity	Provide 1.73 days detention time at 4 mgd rate
20	Lakelet No. 3 aeration	Available for use if lake tends toward an anaerobic condition due to algae die off or spring thaw of ice cover	May also be used as an oxygen supply for the fish in the winter
21	Sand filtration	15 gpm/ft^2	4-1.0 mgd pressure filters in parallel, backwash automatically one at a time
22	Recirculation	During summer, lakes' recirculation proposed through the filtration units (Item 21) for alga removal. Winter recirculation proposed by pumping directly from Lakelet No. 3 to Lakelet No. 1.	

TABLE 18 (continued). DESIGN BASIS - MOUNT CLEMENS CITY-WIDE PROJECT

Item no.	Wastewater facility or process unit	Design basis	Remarks
23	Instrumentation and control	Monitor and operate the retention basin and lakelet treatment facilities	Automatic
24	Emergency outlet from retention basin bypassing lakes system to regional system	4 mgd	Outlet provided in case the lakes system is temporarily 'down'.

pump station at the retention basin site failed. However, each structure would have a new flap gate to keep the river water out of the sewer system. For the next 10 to 15 years, at the end of which the relief sewer program is expected to be completed, overflows at these structures are not expected.

Investigation was made to consider eliminating the existing 19 overflow structures in favor of a single overflow chamber at the main pump station. The interceptor at the main pump station is below the river and the head loss through the tunnel would cause flows to back up in the basements.

The overflow chambers were designed with U-trough similar to those designed for the Demonstration Project with one exception. The weir height on the U-trough was adjustable such that the amount of flow which would surcharge to the dry-weather flow disposal system could be easily field adjusted.

The in-system storage possibility, provided by the combined sewer overflow tunnel interceptor, has been considered in the design of the storage required for combined sewer overflows.

Main Pump Station at the Retention Basin Site

The peak rate of flow was calculated using the Rational Method because of its simplicity and general acceptance. The Detroit 5-year rainfall intensity curve utilized time of concentration of 132 min. The runoff coefficient assumed was 0.5.

The surcharge that would occur in the interceptor tunnel would increase the level in the pump station wetwell, and thus decrease the static lift. Hence the maximum capacity of the pump station was calculated to be 428 cfs. When the relief sewers are constructed, and when the interceptor flow exceeds the pump station capacity, it is expected that the capacity of the station would be increased.

Only one source of power (electrical) has been provided. Detroit Edison expects good reliability, i.e. equal to or better than one, 2-hour outage in 3 years. If power failure occurs, the interceptor system would fill up and some overflows may occur at the existing overflow structures, after storage in the combined sewer overflow interceptor has been filled if it rains when the outage occurs.

Sedimentation-Resuspension Chambers (SRC)

Reference 13, p.122 states:

> "Suspended solids consist generally of between 70% and 90% settleable solids in both sewage systems".

Reference 6, p.49 states:

> "60 percent to 70 percent of BOD and Suspended Solids could be removed with 30 minutes of settling time".

Reference 14, p.14 states:

"settleable solids are of the particle size equal to or greater than 0.18 mm assuming a specific gravity of 1.2".

SRC were designed to provide a minimum of 30 min. of settling time. The peak overflow rate was 0.0058 fps or 3760 gpd/sq ft or 1.77 mm/sec. Assuming specific gravity of 1.2, the peak rate is expected to settle solids 0.15 mm and greater. Thus the SRC have the capability to settle out nearly 90 to 100% of settleable solids.

Resuspension and emptying has been explained earlier under subsection 'Operational Description'.

Retention Basin

The retention basin storage was initially computed, utilizing the procedure used to size Lakelet No. 1 in the Demonstration Project with two exceptions. For the city-wide project it was assumed that storms giving rise to a 1.0 inch accumulation would be collected (instead of 1.5 inch accumulation in the Demonstration Project). Further the runoff coefficient was increased to 0.8.

The retention basin storage was next determined from a mass diagram study of 10 years of rainfall data. The analysis involved plotting 10 years of rainfall records on mass diagrams and determining the most reasonable combination of retention capacity, dewatering rate, and number of emergency overflows from the retention basin to the river. Assuming a retention basin capacity, the dewatering rate was varied to determine the number of overflows per year.

The 32 million gallon retention basin has a maximum emptying period of 7.6 days when full.

For aeration and odor control, it was assumed that the incoming BOD5 would be 75 mg/l. The required aeration equipment capacity was calculated, utilizing the procedure used in the Demonstration Project.

An emergency overflow weir and a controlled outlet to the river have been provided from the chlorination chamber of the retention basin.

Chlorination Basin

It is expected that the maximum duration of an emergency overflow from the retention basin to the Clinton River would be 4 hours. During an overflow condition, the expected pumping rate would be 200 mgd. Assuming a dosing rate of 5 mg/l, the capacity of the chlorinator worked out to be 8000 lbs/day. It is expected that the chlorine consumption per emergency overflow would be 1400 lbs.

Flushing System

The water supply for washing down the SRC is from the chlorination basin, the retention basin, or the river. The water supply for washing down the retention basin is from the chlorination basin or the river.

Sludge Disposal

The resuspended, aerobic sludge from the SRC is settled out in the clarifier at the treatment-park site. Provision has been made for adding flocculating chemicals and phosphorus removal chemicals (if required) prior to settling. The sludge from the clarifier would be conveyed to the existing sanitary sewer and ultimately to the DMWD regional system.

The monitoring system, consisting of a density meter and flow meter, would control the amount of sludge from the combined sewer overflow collection and treatment facility being delivered to the regional system. Laboratory testing would determine the quality of the sludge.

Assuming 1000 million gallons of overflows (4 mgd for 250 days) per year would be treated, the settled solids (70% of 300 mg/l SS) would amount to 1,751,400 lbs/year or 1359 lbs/acre/year.

Final Effluent Criteria

The Michigan Water Resources Commission has established the following effluent criteria for the discharge of the treated combined sewer overflows to the Clinton River:

a. BOD_5 = 10 mg/l as an average with no single values to exceed 15 mg/l.
b. Suspended Solids = 15 mg/l as an average with no single values to exceed 25 mg/l.
c. Oil (Hexane Soluble) = 10 mg/l or no visible film.
d. Coliform = 1000 organisms/100 ml of sample.
e. Phosphorus removal = provisions must be made for chemical additions should such a requirement eventually become necessary.

Lakes System

The Demonstration Project had 3 lakelets, an aerated Lakelet No. 1 acting as a retention basin, Lakelet No. 2, and an aerated Lakelet No. 3. The total minimum detention time was 16 days and the total maximum detention time was 19 days, at 1 mgd flow rate. The city-wide project has an aerated Retention Basin and 3 lakelets, an aerated Lakelet No. 1, partially aerated Lakelet No. 2, and Lakelet No. 3. The total minimum detention time proposed is 8.6 days and the maximum detention time is 12.4 days, at 4 mgd flow rate. The final effluent from the lakes system is expected to be the same, inspite of increasing the flow rate from 1 to 4 mgd, due to additional aeration provided.

The total aeration provided in the Demonstration Project was for 10.8 days. The total aeration period provided in the city-wide project is for 9.5 days minimum and 11.3 days maximum.

It should be noted that the proposed aeration in Lakelet No. 1 would reduce BOD_5 from 50 mg/l to 10 mg/l.

Construction Costs

The construction of the city-wide project has been planned as follows:

Phase I - i) Construction of the combined sewer overflow interceptor tunnel (Contract C-1)

ii) Installation of flap gates and manhole covers at 19 overflow structures (Contract C-2)

iii) Construction of the retention basin including the main pump station (Contract B)

Phase II - i) Modifications to the Demonstration Project lakes system, and modifications to the dry-weather sanitary wastewater treatment plant being abandoned in favor of the regional system (Contract A)

ii) Combined sewer overflow interceptors to convey overflows to the tunnel interceptor (Contract D)

iii) Sanitary and storm sewers (Contract E)

Phase I is under construction and Phase II is planned for 1975/76. Construction costs (actual bids received) breakdown for Phase I is as follows:

Contract		Bid Amount ($)
C-1		5,985,000
C-2		57,600
B		4,585,000
	TOTAL	10,627,600

Construction of Phase II is estimated to cost $2,400,000.

The total project costs (to include engineering, legal, fiscal, administrative, and property and easement acquisition) are estimated to be 125% of the construction costs.

TABLE 19. OPERATIONS AND MAINTENANCE COSTS-MOUNT CLEMENS CITY-WIDE PROJECT

Item	Cost ($)
Personnel (includes fringe benefits)	
. 1 superintendent	20,000
. 1 mechanic	12,500
. 5 operator/technicians @ $11,500	57,500
. shift differential	3,500
. overtime	3,000
. part-time help	2,500
Personnel	99,000
Operation & Maintenance (other than personnel)	
. materials, parts	8,500
. equipment maintenance	6,000
. vehicles (trucks, loader)	5,000
. uniforms, laundry	2,500
. misc. contractual services	1,000
. mechanical supplies, tools	250
. building maintenance	500
Operation & Maintenance (other than personnel)	23,750
Supplies	
. chlorine @ $3.50 per million gallons	3,500
. phosphorus removal @ $16 per million gallons	16,000
. oil, lubricants	600
. laboratory supplies	1,500
. custodial services and supplies	500
Supplies	22,100
Power (Retention Basin & Lakes System) - annual average	18,500
Heating	800
Capital Outlay	5,000
Miscellaneous	1,500
TOTAL	170,650

Operation and Maintenance Costs

The estimated annual operation and maintenance costs for the city-wide project are shown in Table 19.

Assuming that the retention basin is operated approximately 250 days in a year, and the dewatering rate is 4 mgd, the cost of treatment amounts to $171 per million gallons, or $1.28 per 1000 cu ft.

SUGGESTED DESIGN CONSIDERATIONS AND/OR PROCEDURES

It is our intention here to recommend a general concept for application in future combined sewer overflow renovation projects. This general concept (collection and treatment of combined sewer overflows for the multi-purpose use of both the treatment facility and the renovated wastewater) must be adapted to suit each specific situation by the designing engineer. The general planning, design considerations and/or procedures, as suggested hereafter, include some of the most significant concerns for the optimum application of the Mount Clemens Concept. It is not our intention to provide a design manual.

Rainfall-Runoff Analysis

An important part of the design process is an analysis of the rainfall-runoff relationship. The frequency and intensity of storms, basin characteristics, design storm, and occurrences of additional storms before the design storm is over, are some of the considerations in the analysis. References 15 and 16 are excellent studies to review.

Collection System

An existing sewer system investigation should be performed to produce information concerning the actual watershed service area, the degree of separation within the area, the extent of deterioration of the sewer structures, and the probable capacities within the combined sewer system. This information is crucial in the detailed design of interceptors and retention and treatment facilities. Moreover, if infiltration is found to be excessive, a sewer rehabilitation program may become justified in order to reduce dry weather flows.

Many existing combined sewer systems will be found to have limited storm drainage capacity. If local basement flooding is a problem, it is recommended that ponding in the streets be considered to help prevent the combined sewage from backing up into basements.

The combined sewer overflow interceptors should be designed for storm water runoff using a 5 to 10 year storm recurrence interval. The extra capacity in the interceptor, over that which might be capable of delivery by the collection system, would be available as an outlet for the future installation of additional relief storm sewers.

The existing overflow structures, which allow discharge to the watercourse, should normally be maintained in service. The design basis for the capacity of any lift station proposed for the pumpage of storm water should be consistent with the interceptor design; that is, the peak pumping capacity should be equal to, or greater than, the maximum capacity of the interceptors.

For basins that are not complex and have generally 200 acres or less, it is recommended that the design storm runoff be analyzed by the Rational Method. Even though this method has frequently come under academic criticism for its simplicity, no other practical drainage design method has been evolved to a level of general acceptance by the practicing engineer. The Rational Method properly understood and applied can produce satisfactory results for urban storm sewer design. The greatest drawback to the Rational Method is that it normally provides only one point on the runoff hydrograph. When the basins become complex and where subbasins come together, the Rational Method will tend to overestimate the actual flow, which results in oversizing drainage facilities. Another disadvantage of the Rational Method is that with typical design procedures one normally assumes that all of the design flow is collected at the design point and that there is no "carry over water" running overland to the next design point. However, this is not the fault of the Rational Method, but of the design procedure. There must be some modification to the Rational Method, or another type of analysis used, when analyzing an existing system that is underdesigned or when analyzing the effects of a major storm on a system designed for the minor storm.[16] A modified Rational Method is discussed in Reference 17.

Sedimentation Resuspension Chambers

Sedimentation Resuspension Chambers (SRC) are suggested for use in the removal of settleable solids from the combined sewage prior to its overflow into the Retention Basin.

After a storm has subsided, the settled material in the SRC would be resuspended for conveyance directly to the dry weather flow disposal system or with the retained wastewater to the dry weather flow disposal system.

Aerated Retention Basin

Many considerations must be taken into account in order to adequately and economically size a retention basin.

Initially, the storage capacity of the receiving retention basin is designed to hold a designated runoff volume from the facility service area. The service area runoff characteristics should be carefully examined. In our Mount Clemens' project, this volume was based on a 1 inch rainfall over the service area, but assumed that 80% of the volume would reach the sewer system. We recommend, howeve , that 60% of the volume should be considered. Final design of the retention basin should be based on the analysis of rainfall records on mass diagrams and determining the most reasonable combination of retention basin capacity, dewatering rate, and the number of emergency overflows from the retention basin. Reference 11, Chapter 7 - Combined Sewer Overflow Pollution Control Facilities, gives an example of the mass diagram technique utilized in the design of a retention basin.

The determination of an appropriate dewatering rate should be viewed in relationship with the proposed storm water treatment facility (including a possible phosphorus removal process unit) rate and which would dewater the retention basin in sufficient time to provide retention capacity for the next rainstorm.

Following the determination of the most favorable capacity and dewatering rate for the retention basin, the final layout can then be designed, including the supplemental systems for aeration, disinfection, and flushing.

It should be noted that the primary function of the retention basin is to capture and retain combined sewer overflows. The treatment aspect of the retention facility is secondary. It behooves the design engineer to take maximum advantage of the detention time in a retention basin. The provision of mechanical aeration in the receiving-retention basin as an extremely efficacious location for initiating the satisfaction of total BOD is considered, therefore, one of the more important features of the Mount Clemens concept. Under full retention basin conditions, it is anticipated that it would take about 8 days to completely dewater the average retention basin. Even when only partially filled, the time is usually abundant to be conducive toward stabilization. The retention basin should be open to the atmosphere which is not only more appropriate for maximum oxygen transfer, but is substantially less expensive than an enclosed basin. However, land availability and value must be considered in the determination of whether the basin can be left open or whether it must be enclosed to allow use of the land area above the basin.

All flows discharging directly to the river from the retention basin in Mount Clemens are to be disinfected in a chlorination basin prior to discharge. The chlorination facility was incorporated as a means of meeting specific requirements set forth by the State of Michigan. However, the true cost effectiveness of incorporating chlorination facilities at a retention basin site is worthy of further study and evaluation considering: (1) the high degree of sedimentation which is expected in the SRC and retention basin, (2) the aeration of the overflows in the retention basin during the filling period and (3) the frequency of the overflows from the retention basin to the receiving stream.

Relatively efficient settling is expected to occur in the SRC but there will most likely be some fine particulate material carry over into the retention basin. This material may settle in the retention basin and, upon the dewatering of the retention basin, produce an odoriferous condition. It is therefore deemed necessary to provide a flushing system for use in removing the settled material from the retention basin.

Clarifier

Algae was present in the lake system in excessive quantities during the Demonstration Project studies. It is our recommendation, therefore, that a phosphorus removal mechanism be incorporated during the treatment of the overflows at some location between the retention basin and the lake system. It is suggested that a clarifier, receiving flows from the retention basin would most economically supply this function. The sludge from the clarifier could be discharged to the dry-weather flow disposal system, but consideration should be given to the utilization of the phosphorus-laden sludge as a potentially valuable resource.

The city-wide project for Mount Clemens contemplates the use of the existing clarifiers for potential phosphorus removal. Moreover, it is planned to investigate the use of various substances, including the use of water supply filtration plant waste sludge (another current disposal problem), for phosphorus removal in order to economically remove enough phosphorus from the process water to reduce the algae growth in the recreation lake.

Lake System

It is recommended that a three lake system be provided for the final treatment of combined sewer overflows. The first lake should be isolated and provided with submerged aeration. The submerged aeration will eliminate winter maintenance problems and moreover, maximize the treatment efficiency and help produce a high quality effluent that would not be detrimental to fishing and boating activities. The second lake would act as a transition and polishing lake. The third lake would be used for limited recreational activities. Submerged aeration may be advisable in the third lake to insure that a high dissolved oxygen concentration would be maintained. Submerged aeration is also safer if people were to use the lake.

Based on our experience with combined sewer overflows and with aerated flow through lagoons we would suggest that a lake system be designed to provide 7 to 10 days detention time and have a depth of about 10 feet.

Park Site

We strongly recommend the development of a park site which would incorporate the final lake of the treatment facility for recreational purposes. Moreover, we would recommend that a maximum amount of the processed water be used to water the park greenery and to promote verdurous conditions in possible neighboring facilities such as golf courses. However, we are not, at this time, recommending the use of the process effluent for irrigation of vegetation which is meant for human consumption.

We feel that the multi-purpose aspects of such a facility is an important economic consideration and adds significantly to a total water quality management program.

Additional Suggestions

The DMWD has recently completed a study based upon eighteen months of operation of the sewer monitoring and remote control system. The study recommends telemetered advanced warning of rainfall, sewer monitoring, remote control system, and in-system storage. These recommendations (wherever it is appropriate to utilize them) together with the Mount Clemens Concept should be considered as a feasible alternative for the combined sewer overflow problem.

SECTION X

REFERENCES

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SECTION XI

LIST OF PUBLICATIONS

As a result of this Demonstration Project the following publications have been produced:

1. Mahida, Vijaysinh U. Combined Wastewater Collection and Treatment Facility Mount Clemens, Michigan. (Paper presented at the 44th Annual Meeting of the Water Pollution Control Federation, San Francisco. October 10-13, 1971.) 10p.

2. Manual of Operation and Maintenance - Combined Wastewater and Collection Facility, City of Mount Clemens, Michigan. Spalding, DeDecker & Assoc., Inc., Madison Heights, Michigan. April 1972. 26p.

3. Mahida, Vijaysinh U., F.J. DeDecker. Project Engineering Report - Wastewater Collection System and Treatment Facility, City of Mount Clemens, Michigan. Spalding, DeDecker & Assoc., Inc., Madison Heights, Michigan. July 1973. 22p.

SECTION XII

GLOSSARY OF TERMS AND ABBREVIATIONS

BOD_5	Five day biochemical oxygen demand
cfs	Cubic feet per second
Cl_2	Chlorine
coli.	Coliform
cu ft	Cubic feet
DO	Dissolved oxygen
dwf	Dry-weather-flow (A term used in conjunction with combined sewers which describes the flow of sanitary sewage and industrial wastes at a time when storm water runoff is not entering the combined sewer)
°C	Degrees Centigrade
°F	Degrees Fahrenheit
fps	Feet per second
ft	Feet
gal	Gallons
gpd	Gallons per day
gpm	Gallons per minute
lbs	Pounds
mgd	Million gallons per day
mg/l	Milligrams per liter
ml	Milliliters
min	Minutes
NH_3-N	Ammonia in terms of nitrogen

pH	Hydrogen ion concentration
PO_4-P	Phosphate in terms of phosphorus
PPD	Pounds per day
rpm	Revolutions per minute
sec	Seconds
SRC	Sedimentation-Resuspension Chambers
SS	Suspended Solids
sq ft	Square feet
temp	Temperature
VSS	Volatile suspended solids

SECTION XIII

APPENDICES

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TABLE A-1. PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/100ml	Fecal coli. x10^4/100ml
4/1/70	.25	332	120								
	.50										
	.75	204	116								
	1.00	284	88								
	1.25	248	120								
	1.50										
	1.75	492	304								
	2.25	96	48								
	2.50	208	44								
	2.75	229	40								
	3.00	112	68								
	4.00	348	216								
	4.25	80	68								
	5.50	140	112								
	5.75	288	104								
	6.00	268	220								
4/2/70	0.25	520	192	95					6.7		
	0.50	95	47	30					7.0		
	0.75	344	107	65					7.1		
	1.00	326	95	50					7.1		
	1.25	324	86	90					7.0		
	1.50	379	108	90					6.9		
	1.75	421	127	65					7.0		
	2.00	410	128	75					6.9		
	2.25	392	100	90					7.0		

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/100ml	Fecal coli. $x10^4$/100ml
4/2/70	2.50	344	132	85					7.0		
	2.75	300	120	60					7.0		
	3.00	328	124	70					7.0		
	3.25	252	96	60					6.9		
	3.50	268	132	55					7.0		
	3.75	120	52	50					7.1		
	4.00	172	96	40					7.2		
	4.25	196	68	60					7.0		
	4.50	236	84	85					7.1		
	4.75	284	80	80					7.0		
	5.00	260	92	50					7.0		
	5.25	280	104	45					7.0		
	5.50	252	60	60					7.0		
	5.75	316	80	65					6.9		
	6.00	40	16	40					7.0		
4/20/70	0.25	458	300	160	0.8	3.5					
	0.50	20	18	220	0.4	4.5					
	0.75	116	60	165	0.2	6.5					
	1.00	130	66	100	0.2	7.0					
	1.25	162	86	105	0.1	2.0					
	1.50	104	38	135	0.3	2.5					
	1.75	136	56	210	0.3	4.0					
	2.00	108	42	225	0.1	4.0					
	2.25	94	52	160	2.7	12.5					
	2.50	284	234	125	0.2	2.5					

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
4/20/70	2.75	98	48	185	0.1	2.0					
	3.00	58	24	105	0.1	3.5					
	3.25	86	70	160	0.1	2.5					
	3.50	106	84	125	0.5	5.0					
	3.75			145	0.4	3.0					
	4.00			190	1.5	3.5					
	4.25				0.5	6.0					
	4.50				1.5	3.0					
	5.25	838	556								
	5.50	548	362								
	5.75	144	100								
	6.00	106	68								
5/15/70	0.25			245	8.1	16.4			6.4		
	1.00			200	7.5	14.0			6.6		
	1.50			110	4.7	19.5			6.8		
	1.75			20	2.4	15.0			6.7		
	2.25			100	2.9	14.5			6.3		
	2.50			110	1.9	17.5			7.0		
	2.75			125	2.4	5.5			6.8		
	3.00			150	7.5	9.0			7.0		
	3.25			105	2.4	10.0			6.9		
	3.50			195	4.7	3.0			7.0		
6/11/70	0.25	528	392	232	3.3	5.0			6.2		
	1.0	1176	488	297	3.6	6.5			6.3		

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/100ml	Fecal coli. $x10^4$/100ml
6/11/70	1.75	1100	564	227	3.2	12.0			6.4		
	2.50	384	168	129	2.0	23.0			6.4		
	3.00	392	80	138	1.6	8.0			6.3		
	4.00			120	1.7	6.0			6.3		
	4.50	216	272	109	1.7	10.0			6.5		
	4.75	312	284	124	1.8	5.5			6.6		
	5.00	308	300	127	2.0	5.0			6.4		
	5.50			168	1.9	6.0			6.4		
	5.75	656	552	163	1.9	27.0			6.5		
	6.00			147	2.4	11.5			6.5		
6/18/70	0.25	928	372	315							
	0.50	952	152	133							
	0.75	688	113	100							
	1.00	684	136	70							
	1.25	400	92	53							
	1.50	300	92	100							
	1.75	536	80	53							
	2.25	60	28	105							
	2.50	444	76	40							
	2.75	360	40	45							
	3.00	252	28	68							
	3.50	284	44	65							
	3.75	88	16	15							
	4.00	208	40	30							
	4.25	96	20	90							

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/100ml	Fecal coli. $x10^4$/100ml
6/18/70	4.50	76	20	30							
	4.75	160	48	35							
	5.00	116	28	45							
	5.25	88	24	23							
	5.50	80	16	83							
	5.75	88	30	70							
	6.00	84	24	53							
6/24/70	0.25	1160	792	261					6.5		
	.50	760	468	231					6.7		
	.75	192	124	93					6.9		
	1.00	528	196	88					6.9		
	1.25	496	264	89					6.8		
	1.50	408	192	67					6.6		
6/26/70	0.25	340	172	223	1.7	19.5			6.5		
	0.50	328	152	215	0.9	15.5			6.6		
7/8/70	0.25	576	388	285	8.1	3.5			6.3		
	0.50	656	436	325	6.4	4.9			7.0		
7/12/70	0.25			172							
	0.50			134							
7/20/70	0.50			355							
	2.00			300							

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
7/20/70	2.50			228							
	2.75			128							
	3.00			233							
	3.25			143							
	3.75			95							
	4.00			115							
	4.25			120							
	4.75			245							
	5.25			263							
7/29/70	0.50	528	244								
	1.00	416	196								
	1.25	160	100								
	1.50	476	276								
	2.25	228	124								
	2.75	992	468								
8/19/70	0.75	804	328		3.5	3.0			6.8		
	1.50	664	236		2.5	2.5			6.8		
	2.25	200	92		1.4	2.5			6.8		
	3.00	440	172		1.9	2.5			6.9		
	3.75	384	140		1.4	2.0			6.9		
	4.50	244	74		1.4	2.5			6.9		
	5.25	132	44		3.1	6.0			6.9		
	6.00	192	128		5.6	9.0			7.3		

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD$_5$, mg/l	PO$_4$-P, mg/l	NH$_3$-N, mg/l	NO$_3$-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/100ml	Fecal coli. x10^4/100ml
9/18/70	0.50	636	260		0.2	4.0			6.9	0.07	0.09
	0.75	348	144		0.5	2.5			7.0	0.21	0.04
	1.00	236	100		1.0	2.5			7.0	0.17	0.12
	1.25	444	124		0.9	2.5			6.8	0.06	0.24
	2.00	40	24		1.2	1.0			7.5	0.14	0.16
	2.50	172	52		0.3	1.0			7.3	0.09	0.12
	3.00	156	44		2.3	0.5			7.6	0.21	0.09
	3.25	264	76		1.1	1.0			7.5	0.22	0.12
	3.75	184	40		0.8	1.0			7.0	0.04	0.13
	4.25	80	20		0.3	1.0			7.0	0.06	0.02
	4.50	72	32		0.6	0.5			7.0	0.09	0.01
9/27/70	0.25	164	88		0.6	10.5		42.1	6.6	0.34	0.14
	0.50	192	96		0.3	6.0			6.8	0.42	0.22
1/4/71	0.25	736	452	258	8.9	10.9	1.4		6.8	151	107
	0.50	684	392	269	6.1	10.9	1.3		6.8	76	102
	1.00	768	440	272	7.8	10.4	1.6		6.7	49	83
*overflow caused by melting snow	1.75	262	84	257	5.2	8.3	1.2	39.0	6.7	91	53
	2.23	708	400	261	4.0	4.7	1.2		6.7	36	33
	2.50	900	528	281	5.1	5.4	1.2		6.6	58	6.7
	2.75	796	448	275	4.7	5.2	1.2		6.6	32	86
	3.00	640	368	263	4.3	3.4	1.3		6.7	34	52
	3.25	608	312	254	4.8	4.3	1.3	38.4	6.7	39	50
	3.50	584	204	230	4.7	3.9	1.5		7.0	10	53
	5.75	292	104	147	2.7	3.5	4.7		6.9		19

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/100ml	Fecal coli. $x10^4$/100ml
1/4/71	6.00	268	80	117	3.2	3.3	5.7	38.5	7.0		13
1/4/71	0.25	308	164	96	3.6	5.8	1.5		6.8	392	1
	0.50	336	208	91	2.6	5.2	1.3		6.7	336	8
*Second set of samples	0.75	496	392	114	2.7	3.9	1.1		6.7	336	4
	1.00	256	196	100	3.1	3.8	1.3	33.8	6.7	224	4
	1.25	1064	964	100	2.4	4.7	1.1		6.7	238	4
	1.75	1020	936	146	2.9	3.8	1.2		6.7	252	6
	2.00	588	448	122	2.4	4.8	1.1		6.7	182	5
	2.25	1000	828	153	3.6	6.2	1.2	29.6	6.8	238	5
	Recorded Volume of Overflow = 177,065 cu ft										
	Recorded Duration of Overflow = 9.1 hrs										
2/4/71	0.25	44	32	37	1.0	1.0			6.4	21	4
	1.00	48	32	23	1.0	1.0			6.4	24	5
	1.25	164	28		3.2	1.0			6.4	49	5
	4.00	564	324	157	1.8	2.0		11.8	6.4	58	61
	4.50	340	180	60	1.9	0.5			6.4	41	25
	5.00	352	180	67	2.0	0.5			6.3	66	38
	5.75	64	48	7	2.0	1.0			6.4	56	3
	6.00	44	36		1.0	1.0		24.9	6.5	32	9
	Recorded Volume of Overflow = 215,513 cu ft										
	Recorded Duration of Overflow = 15.0 hrs										
2/17/71	0.50	568	324	171	5.2	6.0			7.0	130	40
	0.75	480	260	162	3.9	4.5			7.0	180	30

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD$_5$, mg/l	PO$_4$-P, mg/l	NH$_3$-N, mg/l	NO$_3$-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
2/17/71	1.00	548	308	160	4.0	6.0			6.3		
	1.25	452	244	139	3.2	5.0			6.5	170	0.2
	1.75	380	204	132	2.9	4.5			6.5	160	
	2.00	404	164	114	3.0	3.5			6.6		
	2.25	106	24	53	2.1	0.5			6.4	100	20
	2.75	880	608	172	3.1	5.5			7.0	140	10
	3.00	36	20	60	3.7	8.0		168.0	6.6		
	3.25	320	156	92	2.6	4.0			6.2	160	20
	3.75	308	100	65	2.2	3.0			6.2	280	60
	4.00	264	68	67	1.4	7.0			6.3		
	4.25	376	164	90	1.6	2.5			6.2	150	150
	5.00	196	40	64	1.6	4.0			6.3	620	730
	5.25	180	44	75	2.3	4.0		151.9	6.2		
	5.50	176	48	82	1.9	4.0			6.2	390	430
	5.75	176	60	58	2.0	3.5			6.1	200	160
	6.00	168	68	73	2.1	2.5		103.0	6.1		

Recorded Volume of Overflow = 103,204 cu ft

Recorded Duration of Overflow = 6.1 hrs

2/18/71	0.25	276	140	110	4.4	6.5			6.6		
	0.50	264	96	99	2.8	6.0			6.8		
	0.75	272	84	86	2.0	6.0			6.8		
	1.00	240	92	77	2.7	4.5			6.8		
	1.25	280	106	76	2.4	4.0		88.8	6.7		
	1.50	308	128	95	3.7	4.0			6.7		
	1.75	288	136	60	1.9	4.0			6.7		

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/ 100ml	Fecal coli. $x10^4$/ 100ml
2/18/71	2.00	320	112	47	1.7	3.5			6.8		
	2.25	316	124	57	1.7	3.0			6.6		
	2.75	292	116	82	1.6	4.0		142.9	6.6		
	3.00	244	104	76	2.1	3.5			6.6		
	3.25	216	92	73	1.1	3.0			6.8		
	3.50	216	108	82	1.0	2.5			6.7		
	3.75	164	64	63	0.8	2.5			6.6		
	4.00	180	76	49	1.4	2.5		137.1	6.5		
	4.25	164	64	75	1.2	2.5			6.6		
	4.50	168	68	72	1.1	3.5			6.6		
	4.75	332	196	135	1.1	2.0			6.5		
	5.00	140	60	82	0.9	2.5		88.0	6.4		
	5.25	156	80	84	0.9	2.5			6.4		
	5.50	136	60	82	1.6	3.0			6.5		
	5.75	152	64	84	1.1	2.5			6.4		
	6.00	240	92	126	2.4	7.0		66.7	6.7		
	Recorded Volume of Overflow = 115,350 cu ft										
	Recorded Duration of Overflow = 8.0 hrs										
2/19/71	Recorded Volume of Overflow = 511,982 cu ft										
	Recorded Duration of Overflow = 16.0 hrs										
2/22/71	0.25	368	96	59	1.1	2.5			6.7	2	
	0.50	284	84	24	0.9	3.0			6.4		
	0.75	240	96	10	0.9	4.5			6.7		
	1.00	156	64	24	0.8	4.0			6.7		

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	NO_3-N, mg/1	Oil, mg/1	pH	Total coli. $x10^5$/ 100ml	Fecal coli. $x10^4$/ 100ml
2/22/71	1.25	204	68	26	0.6	4.0		185.5	6.9	1	
	1.75	308	124	69	4.7	8.5			6.8		
	2.00	932	312	134	4.7	9.0			6.9	40	100
	2.75	560	200	67	1.7	2.5			6.9		
	3.00	488	132	49	1.0	1.5			6.8	13	
	3.25	716	192	56	1.5	5.0		164.5	6.8		
	3.75	888	300	94	2.0	2.0			6.7	4	
	4.25	584	120	32	1.2	7.0			6.9		
	4.50	572	132	26	1.6	4.0			6.9	3	
	4.75	724	144	26	0.9	2.5			6.9		
	5.00	652	128	34	1.6	2.0		92.5	6.9		100
	5.25	584	128	49	1.7	2.5			7.0		
	5.50	584	96	38	1.0	2.5			7.0		
	5.75	516	96	31	0.9	2.0			7.0		
	6.00	448	104	26	0.9	2.0		56.5	7.0	3	
	Recorded Volume of Overflow = 67,793 cu ft										
	Recorded Duration of Overflow = 7.1 hrs										
3/6/71	0.50	300	104	74	2.0	3.5			6.0	300	
	0.75	396	152	97	2.4	2.5			5.9		
	1.00	524	236	169	2.9	2.5			6.0	280	
	1.25	428	188	145	3.2	8.5			5.9		
	1.50	296	132	103	2.1	5.0			6.0	160	10
	1.75	376	164	113	2.9	5.0			6.0		
	2.00	344	152	112	2.5	6.0			6.0	160	
	2.25	240	100	89	1.7	5.5			6.1		

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/ 100ml	Fecal coli. $x10^4$/ 100ml
3/6/71	2.50	296	136	107	1.4	5.0			6.2	250	10
	2.75	220	84	79	1.4	3.5			6.1		
	3.00	196	72	51	1.2	5.0			6.2	390	
	3.25	148	60	46	1.2	2.5			6.1		
	3.50	172	84	41	2.0	2.5			6.2	170	
	3.75	148	72	78	1.5	5.0			6.5		
	4.00	204	92	51	1.5	3.5			6.4	220	
	4.25	408	236	145	3.4	4.0			6.1		
	4.50	320	128	89	1.5	5.0			6.3	230	10
	4.75	296	120	63	1.6	3.5			6.3		
	5.00	296	124	99	1.7	3.5			6.3	340	
	5.25	172	76	72	1.0	1.5			6.3		

Recorded Volume of Overflow = 93,086 cu ft

Recorded Duration of Overflow = 10.1 hrs

3/7/71 Recorded Volume of Overflow = 5,059 cu ft

Recorded Duration of Overflow = 3.3 hrs

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/ 100ml	Fecal coli. $x10^4$/ 100ml
3/13/71	0.50	424	148	117	5.7	7.5			6.6	90	54
	0.75	404	124	112	3.8	6.5			6.3	180	43
	1.00	396	132	122	3.2	6.5			6.3	90	18
	1.25	308	104	122	2.8	8.0			6.4	50	28
	1.50	360	124	116	3.5	4.0			6.5	60	49
	1.75	284	88	101	3.2	7.0			6.4	170	41
	2.00	276	84	110	6.7	9.5			6.5	130	34
	2.25	244	68	83	3.9	9.0			6.5	140	220

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
3/13/71	2.50	224	64	80	6.1	9.0			6.1	550	500
	2.75	192	56	71	7.0	10.0			6.5	410	172
	3.00	168	56	75	3.7	9.0			6.6	320	500
	3.25	148	48	75	3.9	11.0			6.6		56

Recorded Volume of Overflow = 13,356 cu ft

Recorded Duration of Overflow = 4.0 hrs

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
3/19/71	0.50	396	212	206	6.5	10.5			7.0	13	30
	0.75	492	244	225	5.7	7.5			6.8	6	54
	1.00	576	232	197	3.7	8.0			6.7	9	53
	1.25	148	56	98		6.0			6.9	6	
	1.50	368	124	130	3.6	6.5			6.8	7	38
	1.75	384	120	128	3.3	6.0			6.7	5	25
	2.00	400	126	120	3.5	6.0			6.8	8	27
	2.25	324	88	103	2.8	6.0			6.8	2	
	2.50	364	96	107	2.4	5.0			6.9	1	
	2.75	412	100	114	2.4	5.0			6.9		
	3.00	396	88	114	2.7	5.5			6.9	3	

Recorded Volume of Overflow = 14,166 cu ft

Recorded Duration of Overflow = 4.7 hours

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
5/19/71	0.50	232	84	140	2.1	6.0	0.9		6.0	200	40
	0.75	444	148	154	4.5	6.0	1.0		6.2	460	170
	1.00	408	172	229	4.4	3.5	0.9		6.1	370	110
	1.50	280	120	192	3.7	3.0	1.2		6.2	460	80
	1.75	308	112	126	3.4	2.5	0.7		6.0	590	100

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/100ml	Fecal coli. x10^4/100ml
5/19/71	2.25	352	172	112	2.6	3.0	0.5		6.1	320	70
	2.50	232	76	97	3.0	2.5	0.4		6.2	230	10
	2.75	220	68	75	2.4	1.5	0.5		6.3	350	150
	3.00	240	104	80	4.3	1.5	0.6		6.3	170	90
	3.25	160	68	80	4.0	1.0	0.7		6.2	230	110
	Recorded Volume of Overflow = 24,284 cu ft										
	Recorded Duration of Overflow = 3.8 hrs										
5/24/71	0.50	568	452	276	6.5	5.3	0.9		6.8	20	50
	0.75	292	220	187	4.5	4.8	0		6.7	70	30
	Recorded Volume of Overflow = 5,059 cu ft										
	Recorded Duration of Overflow = 2.0 hrs										
6/2/71	0.50	612	396	253	10.4	6.5	1.7		6.3	270	110
	0.75	488	268	250	6.5	4.5	1.8		6.5		
	1.00	396	196	219	4.9	4.0	1.4		6.5	180	90
	1.50	120	52	109	2.3	2.0	2.2		6.4		
	1.75	196	60	139	2.6	1.5	1.5		6.4	90	
	2.00	132	48	89	1.3	1.5	3.0		6.3		
	2.25	136	40	80	1.2	1.5	2.4		6.5	60	50
	2.50	84	4	71	1.5	1.5	4.8		6.4		
	2.75	96	48	74	1.4	1.5	2.6		6.3	30	20
	3.00	96	64	68	1.2	1.0	4.6		6.5		
	3.25	116	44	93	1.7	1.5	4.1		6.4	130	20
	3.50	92	60	78	0.9	1.5	4.7		6.5		
	3.75	64	16	63	0.8	1.5	4.3		6.5	20	50

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/100ml	Fecal coli. x10^4/100ml
6/2/71	4.00	68	16	63	0.6	6.5	4.7		6.6		
	4.25	76	36	58	0.9	6.5	4.8		6.7	10	10
	4.50	72	40	64	0.5	6.5	4.3		6.6		
	4.75	88	4	76	1.2	7.0	4.4		6.7	30	20
	5.00	44	4	73	0.6	5.0	4.4		6.7		
6/13/71	0.50	644	364	249	5.6	5.5	8.4		5.9	130	60
	0.75	320	204	215	5.4	8.5	6.2		6.3	120	70
	1.00	336	180	267	4.4	7.0	6.4		6.0	190	70
	1.75	848	488	269	5.2	5.0	10.1		6.2	170	100
	2.25	592	292	226	3.6	5.0	10.3		6.4		240
	3.00	540	284	204	3.4	2.5	8.8		6.5		410
	3.25	284	148	165	1.9	4.0	8.5		6.2	10	70
	3.50	372	176	156	2.0	4.5	8.8		6.4	40	40
	4.00	192	52	116	2.3	3.0	9.9		6.3		180
	4.25	144	44	169	1.5	4.0	7.2		6.3		70
	4.50	136	44	114	1.7	3.0	7.2		6.4		
	4.75	232	120	110	1.2	3.0	6.5		6.4	20	80
	5.00	64	44	124	2.0	10.0	3.3			20	160
	5.25	124	56	128	1.3	4.0	5.5		6.4	20	100

Recorded Volume of Overflow = 23,272 cu ft

Recorded Duration of Overflow = 2.5 hrs

9/28/71	0.25	520	312	125	2.1	44.5	1.0		7.1		1500
	0.50	276	140	830	2.1	34.0	0		6.7		1500
	0.75	172	76	530	1.6	33.0	0		6.9		700

*Hours after start of overflow.

TABLE A-1 (continued). PRE-CONSTRUCTION STUDIES CLEMENS STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/ 100ml	Fecal coli. $x10^4$/ 100ml
9/28/71	1.00	188	92	570	1.3	15.0	0.6		7.0		1000
	1.75	188	80	480	1.1	10.5	0		6.7		800
	2.25	200	104	480	1.7	7.0	0		6.7		1000
	2.50	472	464	480					6.4		400
	2.75	148	108	34	1.4	7.0	0.4		6.7		1800
	3.00	88	76	200	20.0	11.0	0.2		7.3		TNTC**
	3.25	216	164	172	20.0	16.0	0.4		7.3		6000
	3.50			159	22.0	15.5	0.4		7.1		6300
	3.75			169	22.0	16.0	0.3		7.3		5700
	4.00			164	24.0	48.0	0.1		7.0		7600
	4.50	80	76	169	9.0	48.0	0.2		7.1		TNTC
	4.75	60	52	160	12.0	26.0	0.2		7.1		TNTC

*Hours after start of overflow.
**Too numerous to count.

TABLE A-2. PRE-CONSTRUCTION STUDIES SPRUCE STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD$_5$, mg/l	PO$_4$-P, mg/l	NH$_3$-N, mg/l	NO$_3$-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
4/2/70	0.25	536	184	165	2.5	17.5			6.0		
	0.50			135	1.0	19.5			7.0		
	0.75			255	0.8	15.0			7.0		
	1.00	520	508	143	1.2	14.0			7.3		
	1.25			155	2.0	15.5			7.6		
	1.50	332	228	110	0.6	10.0			7.9		
	1.75	140	96	123	0.5	9.0			8.0		
	2.00	728	220		0.6	5.5			6.3		
	2.25	500	460	108	1.0	5.7			5.2		
	2.50			113	0.9	4.5			5.9		
	2.75	204	120	88	0.8	4.6			6.8		
	3.00	352	172	90	2.2	4.0			6.4		
	3.25	468			1.4	16.2			5.9		
	3.50				1.7	18.5			6.0		
	3.75				3.5	14.0			6.6		
	4.00			65	2.6	9.0			6.8		
	4.25				2.4	5.7			6.7		
	4.50				0.5	15.0			6.8		
	4.75				1.0	14.0			6.7		
	5.00				0.8	3.0			6.6		
	5.25				0.3	4.5			7.0		
	5.50				0.4	4.0			7.1		
	5.75				0.2	5.5			6.6		
	6.00				3.8	3.0			6.8		
4/28/70	4.25			58	2.8	12.5			7.1		

*Hours after start of overflow.

TABLE A-2 (continued). PRE-CONSTRUCTION STUDIES SPRUCE STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD$_5$, mg/l	PO$_4$-P, mg/l	NH$_3$-N, mg/l	NO$_3$-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
4/28/70 *Overflow caused by melting snow.	5.50			200	1.2	14.0			7.3		
	5.75			160	0.8	3.0			6.9		
	6.00			180	0.3				7.0		
6/18/70	0.25	184	64	82	7.8				7.4		
	0.50	60	12	55	9.8				7.2		
	1.00	40	2	73	8.1				7.4		
	1.50	268	52	60	8.5				7.2		
	2.00	1472	384	123	4.3				7.1		
	2.25	2140	28	195	3.6				7.1		
	2.75	220	56	50	9.3				7.2		
	3.25	284	112	107	6.0				7.2		
	3.75	272	80	64	4.7				7.2		
	4.00	340	116	95	2.9				7.2		
	4.50	112	36	48	7.3						
	5.25	68	24		2.4				7.3		
	5.50	144	24	66	7.8	2.5			7.5		
	5.75	164	36	55	9.8	3.0			7.4		
	6.00	244	72	58	8.1	5.0			7.3		
6/24/70	0.25	260	204	178					6.3		
	0.50	260	184	170					6.5		
	0.75	716	504	231					6.7		
	1.00	832	340	216					6.4		
	1.25			246					6.6		
	1.50	620	252	240					6.7		

*Hours after start of overflow.

TABLE A-2 (continued). PRE-CONSTRUCTION STUDIES SPRUCE STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/ 100ml	Fecal coli. $x10^4$/ 100ml
6/24/70	2.50	680	568	179					6.0		
	3.00	576	456	174					6.8		
	3.25	580	332	194					6.5		
	3.75	308	208	105					6.3		
	4.00	648	260	98					6.8		
6/26/70	0.25	116	72								
	0.50	80	60								
	0.75	148	88								
	1.00	212	136								
	1.25	188	116								
	1.50	188	108								
	1.75	352	224								
	2.25	160	92								
	2.50	144	92								
7/8/70	1.00	180	132	196	6.3	5.0			7.3		
	1.25	312	200	218	5.2	4.5			6.9		
	1.50	256	176	214	2.7	6.0			6.4		
7/12/70	0.50			185							
	1.00			155							
	1.25			183							
	1.50			102							
	2.25			195							
	2.75			93							

*Hours after start of overflow.

TABLE A-2 (continued). PRE-CONSTRUCTION STUDIES SPRUCE STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/100ml	Fecal coli. $x10^4$/100ml
7/20/70	0.50	352	232		2.7	10.0			6.8		
	2.00	1108	740		2.8	12.5			7.0		
	2.50	276	124		3.0	9.0			7.1		
	2.75	384	188		1.7	14.0			7.2		
	3.00	336	170		2.1	12.5			6.9		
	3.25	327	166		0.8	3.0			7.0		
	3.75	177	76		0.5	2.5			6.5		
	4.00	235	128		0.3	0			6.9		
	4.25	232	100		0.4	0			7.0		
	4.75	549	364		0.3				7.2		
	5.25	521	334		0.8				7.0		
8/19/70	1.50	1212	540		4.5	4.0			6.6		
	2.00	764	216		2.4	2.5			6.6		
	2.25	200	120		2.7	3.0			6.5		
9/18/70	0.25	624	424	87	0.6	2.0			6.8	.07	.0060
	0.50	392	260	44	0.4	2.5			7.0	.09	.0020
	0.75	244	144	54	0.3	5.0			7.3	.16	.0050
	1.50	2948	920	84	0.7	1.5			7.1	.14	.0010
	2.00	1504	468	57	0.5	0.5			7.5	.21	.0130
	2.50	1280	332	41	0.2	3.5			6.6	.27	.0070
	3.25	756	248	39	0.1	7.0			7.0	.01	.0190
	3.50	1408	948	65	0.1	3.0			7.0	.12	.0220
	4.00	488	184	35	0.6	2.0			7.4	.16	.0160

*Hours after start of overflow.

TABLE A-2 (continued): PRE-CONSTRUCTION STUDIES SPRUCE STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/ 100ml	Fecal coli. $x10^4$/ 100ml
9/27/70	2.25	644	164		7.2	12.0			6.3	.12	.0170
	3.50	288	58		2.4	2.5			7.2	.09	.0060
	3.75	300	72		0.2	2.0			7.4	.17	.0050
	4.00	212	58		0.4	10.5			7.6	.24	.0090
	4.50	496	136		0.6	13.5			7.0	.06	.0140
	5.75	60	32		0.5	6.0			7.1	.05	
11/3/70	0.50	408	228	233	2.0	14.0			7.0	240	440
	1.50	340	240	223	3.1	7.5			7.1	280	260
	1.75	452	236	209	3.3	9.0			7.3	340	170
	2.00	480	220	201	3.0	8.5			7.0	520	100
	2.50	756	412	188	4.7	7.0			7.4	160	200
	3.75	628	292	200	7.5	6.0			7.2	730	100
12/19/70	1.25	336	328	124	3.4	10.0	1.6		6.5		
	1.50	636	472	129	4.3	11.5	2.2		6.5		
	1.75	420	284	128	3.4	13.5	2.8		6.5		
	2.00	320	204	111	3.0	10.0	1.9		6.5		
	2.25	284	200	87	3.8	8.5	2.0		6.6		
	2.50	248	160	81	4.2	6.0	1.5		6.5		
	3.25	436	300	85	3.4	7.0	1.8		6.6		
	3.75	260	180	89	3.1	5.0	1.6		6.7		
	4.00	452	348	76	3.9	7.0	1.6		6.7		
	4.25	432	340	100	3.1	7.0	1.8		6.6		

Recorded Volume of Overflow = 21,248 cu ft

Recorded Duration of Overflow = 4.2 hrs

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*Hours after start of overflow.

TABLE A-2 (continued). PRE-CONSTRUCTION STUDIES SPRUCE STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
2/4/71	1.25	256	156	189	1.7	0.5			6.3	300	46
	1.50	220	104	190	1.4	1.0			6.3	100	29
	1.75	224	84	194	1.3	0.5		11.3	6.2	102	31
	3.00	208	28	322	4.3	7.5			6.4	200	78
	3.25	420	304	313	4.3	11.0			6.5	300	86
	3.75	464	288	271	5.2	3.0		24.9	6.5	300	115
	4.00	640	176	283	4.0	5.0			6.5	400	108
	4.50	316	228	267	4.2	3.0			6.5	156	92
	5.00	156	120	203		0.5			6.5	300	84
	5.50	88	64	94	0.6	2.0			6.5	200	58
	Recorded Volume of Overflow = 184,147 cu ft										
	Recorded Duration of Overflow = 22 hrs										
2/17/71	0.25	308	196	100	5.9	11.0			6.5	140	60
	0.50	412	244	171	5.2	10.0			6.5	100	70
	0.75	336	228	141	3.8	6.0			6.7		
	1.00	444	229	153	4.2	6.5			6.6	40	150
	1.25	352	200	131	3.0	5.0			6.4	80	40
	1.50	492	240	139	3.8	5.5		143.6	6.4		
	1.75	392	204	104	3.0	4.5			6.5	110	20
	2.00	328	184	197	2.7	3.5			6.4	120	40
	2.25	416	200	188	3.2	4.0			6.5		
	2.50	644	308	185	4.7	5.0			6.4	120	30
	2.75	568	248	226	3.0	3.0			6.4	100	30
	3.25	436	164	118	2.9	4.0		174.5	6.5		
	3.50	352	120	104	2.3	5.5			6.4	80	30
	3.75	348	116	112	2.1	4.5			6.3	100	50

*Hours after start of overflow.

TABLE A-2 (continued). PRE-CONSTRUCTION STUDIES SPRUCE STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
2/17/71	4.25	320	100	125	2.8	4.5			6.3		
	4.50	272	80	83	2.4	6.0		47.4	6.4	100	10
	4.75	428	272	156	2.0	3.5			6.3	80	20
	5.00			89	2.7	5.0			6.4		
	5.25	888	132	93	1.9	5.0			6.2	140	10
	5.50	328	92	67	1.9	4.0			6.2	160	90
	5.75	168	112	89	2.0	5.0			6.4		
	6.00	352	132	115	2.0	5.5		44.8	6.4	100	80
	Volume of Overflow = 24,283 cu ft										
	Duration of Overflow = 4.2 hrs										
2/18/71	0.50	212	116	150	6.5	12.5			6.9		
	0.75	180	88	139	5.2	10.5			6.9		
	1.00	308	136	133	4.4	6.0			6.9		
	1.25	328	148	124	3.7	5.0			6.7		
	1.50	428	160	141	3.2	5.0		65.0	6.9		
	1.75	476	204	140	3.1	4.0			6.8		
	2.50	416	124	110	1.9	3.0			6.8		
	2.75	288	92	115	2.2	2.5			6.8		
	3.00			148	2.0	2.0			6.7		
	3.25	268	116	110	2.4	3.5		99.1	6.7		
	3.50	184	68	118	1.7	4.0			6.7		
	3.75	226	108	87	1.7	4.0			6.6		
	4.00	212	100	108	3.2	2.5			6.6		
	4.25	228	100	114	2.0	2.5			6.7		
	4.50	188	72	85	2.0	2.0		132.3	6.7		
	4.75	172	60	80	1.8	2.5			6.7		

*Hours after start of overflow.

TABLE A-2 (continued). PRE-CONSTRUCTION STUDIES SPRUCE STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil, mg/l	pH	Total coli. $x10^5$/ 100ml	Fecal coli. $x10^4$/ 100ml
2/18/71	5.00	220	72	88	1.8	1.0			6.7		
	5.25	104	84	86	2.0	2.0			6.7		
	5.75	212	72	85	1.6	2.5			6.7		
	6.00	176	92	144	5.7	14.0		107.6	6.8		
	Recorded Volume of Overflow = 50,084 cu ft										
	Recorded Duration of Overflow = 8.0 hrs										
2/19/71	Recorded Volume of Overflow = 237,772 cu ft										
	Recorded Duration of Overflow = 16.0 hrs										
2/22/71	0.50	340	88	32	0.8	1.0			7.5	1	
	1.00	284	80	23	0.6	2.5			7.0		
	1.25	220	64	20	0.6	2.5			6.9		
	1.50	208	64	24	0.6	2.5			7.0		
	1.75	232	76	24		5.0		162.7	6.8	2	
	2.50	980	200	106	0.6	12.0			6.7		
	2.75	624	160	89	4.7	8.0			6.7	2	
	3.00	920	228	92	2.9	7.5			7.0		
	3.50	1184	248	124	2.4	7.5			6.9	8	
	4.00	528	116	73	2.8	6.0			6.8		
	4.25	608	136	41	1.9	7.5		166.2	7.1	1	
	4.50	752	156	42	1.4	6.0			6.8		
	5.00	800	160	49	1.6	6.0			7.3	2	
	5.25	576	108	48	1.5	6.0			7.1		
	5.50	400	92	33	1.3	5.0			7.1		
	5.75	388	64	39	0.8	5.0			6.9		

*Hours after start of overflow.

TABLE A-2 (continued). PRE-CONSTRUCTION STUDIES SPRUCE STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	NO_3-N, mg/l	Oil mg/l	pH	Total coli. $x10^5$/ 100ml	Fecal coli. $x10^4$/ 100ml
2/22/71	6.00	272	32	24	0.5	6.5		141.5	7.0		
	Recorded Volume of Overflow = 24,283 cu ft										
	Recorded Duration of Overflow = 8.7 hrs										
3/6/71	0.75	160	44	53	2.4	9.0			5.8	15	100
	1.00	420	136	120	2.9	8.0			6.0		
	1.25	296	104	86	2.8	10.0			5.9	15	100
	1.50	376	120	84	2.7	8.0			6.0		
	1.75	820	144	180	3.5	7.5			6.0	9	
	2.25	240	92	59	1.8	5.0			6.0	14	
	2.50	380	152	71	2.0	5.5			6.1		
	2.75	284	116	62	1.9	5.5			6.1	15	100
	3.25	252	92	55	2.1	7.0			6.1	13	100
	3.50	280	120	62	1.9	7.0			6.2		
	3.75	284	120	55	1.9	6.5			6.2	7	
	4.25	256	96	40	1.8	7.5			6.1	12	100
	4.50	160	60	34	1.2	5.5			6.2		
	4.75	248	36	46	1.0	5.5			6.0	6	
	5.00	232	88	39	0.9	4.5			5.9		
	5.25	272	104	49	1.2	6.0			6.0	6	
	5.50	112	52	18	0.4	4.0			6.0		
	5.75	168	72	38	0.6	4.0			6.0	18	
	6.00	228	124	62	0.9	2.0			6.1		
	Recorded Volume of Overflow = 11,130 cu ft										
	Recorded Duration of Overflow = 8.7 hrs										

*Hours after start of overflow.

TABLE A-2 (continued). PRE-CONSTRUCTION STUDIES SPRUCE STREET DRAIN OVERFLOW SAMPLING DATA

Sampling date, mo/day/yr	Time of sample, hr*	SS, mg/l	VSS, mg/l	BOD$_5$, mg/l	PO$_4$-P, mg/l	NH$_3$-N, mg/l	NO$_3$-N, mg/l	Oil, mg/l	pH	Total coli. x10^5/ 100ml	Fecal coli. x10^4/ 100ml
3/7/71	Recorded Volume of Overflow = 3,035 cu ft										
	Recorded Duration of Overflow = 0.6 hr										
3/13/71	Recorded Volume of Overflow = 21,046 cu ft										
	Recorded Duration of Overflow = 3.0 hr										
5/19/71	Recorded Volume of Overflow = 34,402 cu ft										
	Recorded Duration of Overflow = 3.0 hr										

*Hours after start of overflow.

TABLE B-1. POST-CONSTRUCTION STUDIES SAMPLING DATA

KEY

Sampling location no.	Sampling location description
4	Combined sewer overflow pump station discharge
5	Microstrainer influent
6	Microstrainer effluent
7	Midpoint of Lakelet No. 2
8	Effluent from Lakelet No. 2
9	Sand Filter Influent
10	Sand Filter Effluent

An asterisk after the sampling date, e.g. 12/26/72*, indicates that the wastewater was recirculated from Lakelet No. 3 to Lakelet No. 2, with or without passing through the microstrainer, during prolonged dry periods.

TABLE B-1. POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	pH	DO, mg/l	Cl_2, mg/l	Total Coli / 100ml	Fecal Coli./ 100ml	Temp., °F
8/10/72	5	26	12	10	2.3	18.5	7.4					
	6	22	10	8	2.2	18.5	7.5					
	7										171	
8/11/72	5	27	12	12	3.3	23.5						
	6	26	11	12	3.1	23.5						
8/12/72	5	26	25	13	2.1	10.0	7.5					
	6	9	6	12	2.0	10.0	7.5					
8/13/72	5	48	9	8	2.2	6.0	7.2					
	6	48	9	8	2.2	6.0	7.2					
8/14/72	5	38	8	8	0.2	0.5	7.6					
	6	38	7	3	0.2	0.5	7.6					
8/15/72	4	586	102	60	2.5	0.8	6.8					
	5	40	24	6	2.2	1.5	7.4					
	6	40	22	6	1.9	1.5	7.4					
8/16/72	4	136	22	38	0.9	1.0	7.1					
	5	45	16	6	2.4	0.8	7.3					
	6	34	12	5	2.4	0.7	7.4					
	8	45	14	10	1.7	0.9	7.2					
8/17/72	5	21	8	5	3.5	6.5	7.6					
	6	21	8	5	3.5	6.5	7.6					
	7							7.1	0.8	0	0	
	9	11	3	3	1.4	2.0	7.7	7.6				
	10	11	3	3	1.4	2.0	7.4					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD$_5$, mg/l	PO$_4$-P, mg/l	NH$_3$-N, mg/l	pH	DO, mg/l	Cl$_2$, mg/l	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp. °F
8/18/72	5	22	10	6	0.8	7.0	7.5					
	6	18	8	5	0.6	7.0	7.5					
	8	58	15	4	1.5	6.2	7.0	8.0				
	9	12	3	3	0.4	3.8	7.8	9.1				
	10	5	1	3	0.4	2.0	7.5	8.8				
8/20/72	5	27	10	5	2.4	9.5	7.3					
	6	22	8	5	2.4	9.5	7.2					
	9	16	5	3	1.1	4.2	7.0					
	10	4	2	3	1.1	2.5	6.9					
8/21/72	5	50	14		2.0	9.0	7.6					
	6	14	2		2.0	8.5	7.7					
	8	9	3		1.8	6.4	7.4					
	9	9	3		0.9	4.5	7.8			21	5	
	10	9	3		0.9	2.8	7.6					
8/22/72	5	36	14		2.2	7.5	7.4					
	6	31	14		1.7	7.5	7.5					
	9	10	2		1.6	5.0	7.8					
	10	3	1		1.5	5.0	7.5					
8/23/72	9	8	4		2.4	8.5	7.8					
	10	6	4		2.2	5.0	7.6	6.3				
8/24/72	4	200	84	86	0.9	9.0	7.1					
	5	10	5	12	0.5	5.2	7.0					
	6	6	0	12	0.5	5.2	6.9					
	7							9.8	2.0	0	0	

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD$_5$, mg/1	PO$_4$-P, mg/1	NH$_3$-N, mg/1	pH	DO, mg/1	Cl$_2$, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
8/25/72	5			10	2.4	6.3	7.5					
	6			9			7.6					
	7							6.9	1.3	0	0	
	8	2	1	6	1.2	3.8	7.3	6.5		0	0	
8/26/72	5	38	12	2	1.7		7.6					
	6	38	10	2	1.5		7.7					
8/27/72	5	13	9	28	2.9	4.0	7.6					
	6	13	8	28	2.9	3.3	7.7					
8/28/72	4	382	276	102	7.8	8.5	7.3					
	5	41	18	17	3.2	7.5	7.6					
	6	41	18	16	3.0	7.0	7.6					
8/29/72	5	14	9	48	2.0	8.5	6.6					
	6	8	4	48	2.0	8.5	7.0					
	8	6	4	4	3.2	4.3	7.0	6.3		0	0	
8/30/72	5	33	17	12	3.9	8.5	7.3					
	6	33	14	10	3.9	8.0	7.4					
8/31/72	4	59	33	43	4.4	1.7	7.2					
	5	19	9	9	3.8	1.8	7.0					
	6		9	8	3.4	1.7	7.3					
9/1/72	5	16	7	8	2.9	1.0	7.4					
	6	16	7	8	2.8	1.0	7.4					
9/2/72	5	20	11	8	3.7	3.5	7.2					
	6	19	10	8	3.7	3.5	7.3					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
9/3/72	5	20	11	9	3.1	0.9	7.4					
	6	17	8	9	3.1	0.9	7.1					
9/4/72	5	19	7	51	3.0	11.5	7.2					
	6	19	7	48	3.0	10.5	7.5					
9/5/72	5	22	12		3.7	17.0	7.6					
	6	17	9		3.2	17.0	7.7					
9/6/72	5	22	13		3.7	17.0	7.5					
	6	22	13		3.7	17.0	7.6					
9/7/72	5	20	11		3.3	17.5	7.5					
	6	20	11		3.1	17.5	7.2					
	9	13	4		1.6	16.0	6.8					
	10	6	2		1.6	13.0	6.7					
9/8/72	5	72	12	14	2.4	15.0	6.2					
	6	46	6	11	1.8	12.5	6.5					
	7							8.3			76	
	9	23	5	6	1.6	16.0	6.8					
	10	11	3	5	1.6	13.0	6.7					
9/9/72	5	31	11	3	2.4	15.0	6.2					
	6	29	10	3	1.8	15.0	6.5					
	9	13	9	3	1.4	12.0	6.8					
	10	13	3	2	1.2	7.5	6.7					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
9/10/72	4			235								
	5	30	6	9	4.2	21.5	6.9					
	6	20	7	9	3.9	18.0	7.4					
	9	33	6	12	2.7	6.0	7.4					
	10	18	4	12	2.7	5.0	7.3					
9/11/72	5	31	13	8	3.5	9.0	7.2					
	6	30	10	5	3.2	9.0	7.4					
	9	29	6	3	2.8	5.0	7.2					
	10	14	4	3	2.0	0.5	7.1					
9/12/72	5	35	8	5	2.7	8.0	7.0					
	6	35	5	4	2.7	8.0	7.4					
9/13/72	4	1024	190	27	2.9	0.5	7.1					
	5	96	24	11	3.2	5.5	7.3					
	6	87	21	11	2.5	3.0	7.4					
9/14/72	5	30	12	11	5.3	7.4	7.3					
	6	30	12	10	5.1	6.8	7.4					
9/15/72	5	20	10	10		5.3	7.2					
	6	18	9	9		5.3	7.5					
9/16/72	5	18	10	11	1.0	6.2						
	6	18	10	11	1.0	6.2						
9/17/72	5	20	10	10	3.1	6.4	7.5					
	6	20	8	9	3.1	6.4	7.7					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	pH	DO, mg/l	Cl_2, mg/l	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
9/18/72	4	72	24	60	3.7	9.0	7.6					
	5	27	10	10	4.0	7.2	7.5					
	6	26	9	9	3.8	7.2	7.7					
9/19/72	5	19	5	10	2.9	6.0	7.9					
	6	19	5	6	2.4	6.0	7.8					
	9	21	4	2	2.6	5.5	7.7					
	10	15	4	1	2.5	4.5	7.6					
9/20/72	5	21	9		3.9	7.0	7.5					
	6	21	9	4	3.3	6.6	7.6					
	9	16	5		3.0	4.2	7.9	7.0				
	10	7	3		2.6	3.2	7.5	8.1				
10/2/72	4	198	81	38	2.1	7.0	7.0					
10/4/72	5	44	1	2	2.9	9.0	7.2					
	6	38	1	2	2.9	9.0	7.3					
10/5/72	5	24	14	14	2.5	11.5	7.4					
	6	20	11	13	2.5	11.5	7.5					
10/6/72	4	31	16	32	5.1	11.5	7.3					
	5	23	10	14	3.6	11.5	7.4					
	6	23	10	14	2.4	11.5	7.4					
10/7/72	5	24	14	10	3.5	7.7	7.4					
	6	20	11	10	3.0	7.2	7.4					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
10/8/72	4	60	46	75	3.8	11.0	7.3					
	5	15	12	17	2.9	9.0	7.3					
	6	3	1	15	2.6	9.0	7.4					
10/9/72	5	27	16	18	3.5	12.0	7.4					
	6	17	11	13	3.0	12.0	7.5					
10/10/72	4	20	14	25	5.7	26.0	7.0					
	5	25	15	11	4.8	11.0	7.2					
	6	17	10	11	4.3	11.0	7.2					
10/13/72	5	40	16	8	5.6	10.5	7.3					
	6	32	13	8	4.4	10.5	7.3					
10/14/72	5	42	12	2	1.4	13.0	7.0					
	6	32	11	2	1.4	12.0	7.5					
10/15/72	5	35	16	8	4.4	13.5	7.3					
	6	35	10	7	4.3	13.5	7.5					
10/16/72	5	74	16	3	3.2	15.0	7.3					
	6	50	12	3	3.0	15.0	7.5					
10/17/72	5	43	12	3	2.4	32.0	6.8					
	6	31	11	2	2.4	26.5	7.1					
10/18/72	4	51	21	25	0.3	12.5	7.1					
	5	52	21	6	3.6	2.0	7.0					
	6	52	10	6	3.6	2.0	6.9					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD$_5$, mg/l	PO$_4$-P, mg/l	NH$_3$-N, mg/l	pH	DO, mg/l	Cl$_2$, mg/l	Total Coli./100ml	Fecal Coli./100ml	Temp., °F
10/19/72	4	56	12	15	4.4	16.5	7.2					
	5	48	14	8	1.2	11.5	6.9					
	6	48	10	8	0.6	11.5	7.3					
	7								5.0			
10/20/72	4	43	16	67	3.5	14.5	6.9					
	5	30	14	16	3.3	15.5	7.2					
	6	23	10	16	3.3	15.5	7.0					
	7							10.0	3.0			
10/21/72	5		9	21		13.0						
	6	10	9	14		14.0						
10/22/72	5	22	10	11	5.0	10.0	7.4					
	6	15	9	11	5.0	9.5	7.4					
10/23/72	4	146	29	102	2.9	4.5	7.4					
	5	18	7	15	2.6	4.0	7.7					
	6	14	5	1	2.1	4.0	7.6					
	7							4.5	0.1			54
	8	24	14	11	3.5	7.0	6.6	1.6				52
	9	42	9	16	5.1	4.7	7.5	4.9				
	10	42	9	15	5.1	3.7	7.5	6.9				
10/24/72	5	27	12	12	2.9	3.0	6.4					
	6	27	12	8	2.1	3.0	7.4					
	9	45	11	7	2.8	7.0	7.6					
	10	27	10	7	2.8	5.5	7.7					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
10/25/72	5	21	10	6	2.0	5.5	7.0					
	6	21	10	6	1.6	5.5	7.2					
	9	32	9	2	4.2	14.5	7.4					
	10	9	3	2	3.8	12.5	7.3					
10/26/72	5	20	13	4	1.7		7.1					
	6	16	11	4	1.7		7.1					
	9	33	15	3	8.8		7.1					
	10	13	11	1	6.6		7.1					
10/27/72	5	20	10	5	2.7	5.5	7.4					
	6	17	8	5	2.7	5.5	7.6					
	9	30	8	2	5.2	2.8	7.7					
	10	11	5	2	4.5	2.5	7.7					
10/28/72	5	33	15	5	3.1	4.5	7.3					
	6	24	10	5	2.4	4.5	7.3					
	9	34	8	4	2.2	6.0	7.2					
	10	9	3	2	2.1	6.0	7.2					
10/29/72	5	33	18	12		1.1	7.4					
	6	28	14	11	2.8	1.1	7.5					
	7							5.2	1.0			53
	8	20	9	5	2.6	5.0	7.0	6.6				52

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
10/30/72	5	25	11	10	2.2	5.5	7.0					
	6	25	10	10	1.3	5.5	6.7					
10/31/72	4	196	71	67	4.3	7.5	7.4					
	5	20	13	11	2.8	5.5	7.2					
	6	20	13	11	2.7	5.5	7.2					
11/1/72	4	112	28	120	1.7	8.5						
	5	23	13	15	2.0	10.0						
	6	18	10	14	1.6	10.0						
11/2/72	4	27	18	67	3.1	6.0	6.7					
	5	38	27	20	3.3	7.0	6.8					
	6	26	15	5	2.8	5.0	6.9					
11/3/72	5	42	24	16	3.4	2.5	6.9					
	6	32	18	16	3.3	2.5	6.9					
	7							4.6	0.10		4800	54
	8	6	5	9	2.9	7.0	6.8			0	0	
	9	34	8	8	5.6	3.2	6.9					
	10	14	5	8	3.8	2.0	6.7					
11/4/72	4	26	18	60	3.1	9.5	7.1					
	5	25	13	13	5.2	5.5	7.0					
	6	16	12	12	5.2	5.5	7.1					
	9	26	8	7	2.4	7.0	6.9	9.3				
	10	11	5	7	2.4	7.0	6.9	9.3				

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	pH	DO, mg/l	Cl_2, mg/l	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
11/5/72	5	25	11	14	3.8	6.5	6.9					
	6	17	6	12	3.0	6.5	7.0					
	7							6.2	4.0	0	0	54
	8	16	6	5	3.2	6.0	6.8	6.8			230	50
	9	38	11	8	2.3	6.0	7.0	8.5				
	10	13	4	7	2.2	6.0	6.9	8.7				
11/6/72	5	28	12	15	4.7	6.5	7.3					
	6	24	11	14	4.5	6.5	7.5					
	7							7.3	3.0			54
11/7/72	4	166	80	95	3.7	9.0						
	5	50	27	19	4.2	6.0						
	6	50	25	16	3.6	5.5						
	8	36	25	8	3.0	5.5		7.6				
11/8/72	5	30	16	14	3.6	5.5						
	6	28	14	14	2.5	5.0						
	8									2120	122	
	9	16	5	7	2.4	6.0						
	10	10	5	7	2.2	5.5						
11/9/72	5	31	16	10	3.8	4.0						
	6	28	13	10	3.8	3.5						
	9	16	3	5	3.6	13.5						
	10	2	2	3	3.1	9.0						
11/12/72	4	22	10	18	3.1	9.0	7.2					
	5	42	14	8	2.4	7.0						
	6	35	14	8	2.4	6.5						

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
11/13/72	5	48	18	13	4.4	7.5	6.6					
	6	35	12	12	3.6	7.0	6.6					
11/14/72	4	38	20	30	3.4	5.0	6.6					
	5	47	20	17	4.6	4.5	6.5					
	6	24	12	11	4.6	4.5	6.6					
11/16/72	9	26	6	5	2.5	13.0	7.2					
	10	18	6	3	1.7	10.5	7.2					
11/17/72	5	26	12	17	2.7	5.5	6.5			860	14	
	6	25	10	7	2.4	5.0	6.6					
	8	20	7	4	2.3	13.0	7.2					
	9	14	6	3	2.5	6.5	6.5					
	10	14	6	2	2.5	6.5	6.5					
11/18/72	5	28	15	15	2.3	7.0	6.9					
	6	22	10	11	1.6	6.5	6.8					
	9	18	8		1.7	6.5	6.8					
	10	18	5		1.7	5.0	6.8					
11/19/72	5	23	10	10	2.1	6.0	6.9	6.6	4.0	TNTC[a]	540	44
	6	10	6	6	2.0	6.0	6.9					
	7											
	9	16	5	4	1.5	6.0	6.9					
	10	8	4	4	1.5	5.5	6.9					

[a] Too numerous to count.

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	pH	DO, mg/l	Cl_2, mg/l	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
11/20/72	5	50	18	11	2.4	7.0	6.6					
	6	10	5	8	1.6	6.5	6.8					
	8	12	8	4	1.3	6.0	6.7	5.9		20	2	42
	9	11	4	7	1.7	6.0	6.8					
	10	10	4	5	1.7	5.5	6.8					
11/21/72	4	99	51	80	2.4	9.5	7.2					
	5	60	19	11	3.0	7.0	6.8					
	6	25	7	8	2.4	6.5	6.9					
	9	24	4	7	1.8	6.0	6.8					
	10	18	4	5	1.0	6.0	6.8					
11/22/72	5	46	17	10	2.6	7.0	6.6					
	6	31	16	10	2.6	7.0	6.8					
	9	48	11	13	2.9	8.0	6.8					
	10	20	6	8	1.2	6.0	6.8					
11/23/72	5	90	24	13	2.4	9.0	7.0					
	6	42	9	12	2.1	9.0	7.0					
11/24/72	5	45	12	10	1.9	7.5	6.8					
	6	36	12	10	1.7	7.5	6.4					
11/25/72	5	23	8	5	1.5	5.5	6.8					
	6	23	8	5	1.4	5.5	6.8					
11/26/72	4	98	50	68	2.0	7.0	6.8					
	5	27	5	7	1.0	7.0	6.9					
	6	27	5	7	0.8	6.5	7.0					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD$_5$, mg/1	PO$_4$-P, mg/1	NH$_3$-N, mg/1	pH	DO, mg/1	Cl$_2$, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
11/27/72	5	44	12	16	2.0	10.0	6.4					
	6	31	8	15	2.0	10.0	6.6					
11/28/72	5	63	13	14	2.7	5.5	6.6					
	6	16	9	14	2.6	5.0	6.7					
11/29/72	5	20	8	16	2.7	4.5	6.7					
	6	20	8	2	2.5	4.0	6.7					
	8	16	6	8	2.3	5.0	6.6			2	0	35
11/30/72	5	25	6		1.8	4.0	6.1					
	6	25	6		1.6	4.0	6.7					
12/2/72	5	54	44		1.6	4.5	7.0					
	6	15	14		1.0	4.5	7.1					
12/3/72	5	28	7	34	2.2	2.2	7.3					
	6	25	7	33	1.9	2.2	7.4					
12/4/72	5	93	28	87	2.1	6.0	7.1					
	6	70	21	65	2.1	6.0	7.2					
	8	3	1		2.1	2.4	7.2	9.3				
12/5/72	5	114	27	12	2.5	5.5	7.2					
	6	98	21	10	2.5	5.0	7.2					
12/6/72	4	231	91		2.4	17.5						
	5	100	29	14	1.8	16.0						
	6	64	23	14	1.7	16.0						
12/8/72	9	43	10	4	2.3	8.5	7.3					
	10	26	5	4	1.4	8.5	7.2					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	pH	DO, mg/l	Cl_2, mg/l	Total Coli./100ml	Fecal Coli./100ml	Temp., °F
12/9/72	9	22	4		2.0	6.0	7.1					
	10	22	4		1.8	6.0	7.2					
12/14/72	5	72	22	33	1.4	5.5	6.6					
	6	50	18	30	1.2	5.0	6.6					
12/15/72	4	168	54	11	2.9	10.5	6.8					
	5	26	17	28	1.7	10.5	6.7					
	6	26	17	28	1.4	10.5	6.8					
12/16/72	5	28	16	16	1.7	10.5	6.2					
	6	28	14	16	1.7	10.5	6.3					
12/17/72	5	32	19	16	1.4	5.0	6.9					
	6	26	11	16	1.4	5.0	6.9					
12/18/72	5	22	9	20	1.7	7.0	6.8					
	6	17	6	15	1.7	7.0	6.8					
12/19/72	5	15	10	17	1.5	7.5	6.9					
	6	15	10	17	1.3	6.5	6.9					
12/20/72	5	59	32	28	2.4	8.5	7.0					
	6	20	10	22	1.9	8.5	7.1					
12/21/72	5	30	15	21	2.1	8.5	6.9					
	6	30	15	21	2.0	6.5	6.9					
12/22/72	5	47	16	22	2.3	7.0	6.9					
	6	47	16	20	2.0	7.0	6.9					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	pH	DO, mg/l	Cl_2, mg/l	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
12/23/72	5 6	40 40	11 11	16 16	1.9 1.9	6.4 6.4	6.8 6.9					
12/24/72	5 6	118 83	34 21	34 26	16.0 15.5	8.5 7.5	6.9 6.9					
12/25/72	4 5 6	422 190 190	112 56 56	420 38 38		33.0 32.0 30.0	6.7 6.8 6.9					
12/26/72*	5 6	41 32	8 2	28 28		13.0 13.0	6.8 6.8					
12/27/72*	5 6	24 24	8 8	19 19	2.0 1.1	5.5 5.5	6.8 6.8					
12/28/72*	5 6	28 28	7 7	11 6	1.7 1.7	7.0 7.0	6.8 6.9					
12/29/72*	5 6	24 24	10 10	16 16	1.7 1.1	5.5 5.5	6.8 6.9					
12/30/72*	5 6	29 29	6 6	18 16	1.6 1.6	7.0 6.0	6.8 6.6					
12/31/72	4 5 6	1014 63 63	288 26 26	 16 15	6.1 0.8 0.8	7.0 3.5 3.0	6.8 6.8 6.9					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	pH	DO, mg/l	Cl_2, mg/l	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
1/1/73	5	66	15	13	0.8	4.0	6.8					
	6	65	15	11	0.7	2.5	6.8					
1/2/73	5	45	14	17	1.0	3.0	6.9					
	6	45	14	15	1.0	3.0	6.9					
1/3/73	5	74	14	19	1.1	3.5	6.8					
	6	52	7	15	1.1	3.5	6.8					
	9	21	4	13	1.7	6.0	6.8					
	10	18	4	13	1.6	6.0	6.9					
1/4/73	5	68	16	11	1.4	2.5	6.8					
	6	68	15	8	1.3	2.5	6.8					
	9	20	9	11	1.9	5.5	6.9					
	10	16	8	11	1.7	5.5	6.9					
1/5/73	5	57	10	15	1.4	3.0	6.8					
	6	57	10	14	1.3	3.0	6.8					
	9	19	5	9	1.8	5.0	6.8					
	10	14	5	9	1.3	5.0	6.8					
1/6/73	5	75	21	27	1.3	5.0	6.7					
	6	41	8	11	1.3	5.0	6.7					
	9	18	6	8	1.7	3.0	6.7					
	10	16	6	6	1.7	3.0	6.7					
1/7/73	5	38	12	22	1.6	3.0	6.8					
	6	28	9	15	1.5	3.0	6.8					
	9	18	9	12	1.7	5.0	6.8					
	10	18	9	11	1.7	5.0	6.8					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD$_5$, mg/1	PO$_4$-P, mg/1	NH$_3$-N, mg/1	pH	DO, mg/1	Cl$_2$, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
1/8/73	5	40	11	15	0.8	3.5	7.0					
	6	32	11	14	0.8	3.0	7.0					
1/9/73	5	31	10	15	1.5	2.5	6.7					
	6	31	4	13	1.5	2.5	6.7					
1/10/73*	5	24	7	11	2.0	3.0						
	6	24	7	9	2.0	2.5						
1/11/73	5	46	15	23	1.9	4.0	6.8					
	6	37	12	20	1.8	4.0	6.7					
1/12/73	5	32	13	15	1.8	4.0	6.6					
	6	32	13	12	1.8	4.0	6.7					
1/13/73	5	26	10	23	2.3	5.0	6.6					
	6	26	10	19	2.0	5.0	6.7					
1/14/73*	5	29	8	10	2.0	4.0	6.6					
	6	19	3	10	2.0	4.0	6.6					
1/15/73*	5	19	6	16	2.3	2.5	6.6					
	6	19	6	13	2.3	2.5	6.6					
1/16/73*	5	16	6	10	2.3	5.0	6.8					
	6	15	5	8	2.1	4.0	6.8					
1/17/73*	5	20	6	24	8.1	4.0	6.8					
	6	17	5	17	7.5	4.0	6.8					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	pH	DO, mg/l	Cl_2, mg/l	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
1/18/73*	5	17	7	13	6.5	3.5	6.7					
	6	12	6	10	6.5	3.5	6.7					
1/19/73*	5	16	6	24	5.2	5.0	6.7					
	6	14	6	10	5.2	5.0	6.7					
1/20/73*	5	14	3	4	5.7	5.0	6.7					
	6	11	3	3	5.7	5.0	6.7					
1/21/73*	5	17	5	67	7.5	3.0	6.5					
	6	14	[illegible]	42	7.5	3.0	6.7					
1/22/73*	5	29	6	18	6.1	4.0	6.7					
	6	23	6	18	6.1	4.5	6.8					
1/23/73*	5	11	2	16	5.6	9.0	6.8					
	6	11	2	16	0.7	9.0	6.8					
1/24/73*	5	33	15	33	5.4	5.0	6.8					
	6	33	15	33	5.2	5.0	6.8					
1/25/73*	5	7	4	8	5.4	2.5	6.8					
	6	7	4	8	5.4	2.5	6.8					
	7							8.2	3.0	0	0	
	8	7	2	3	5.9	3.0	6.8	8.8		0	0	33
1/26/73*	5	14	8	11	4.2	3.0	6.8					
	6	10	6	6	4.2	2.5						
1/27/73*	5	12	6	-	2.9	2.5	6.7					
	6	10	4	-	2.9	2.5	6.7					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD$_5$, mg/l	PO$_4$-P, mg/l	NH$_3$-N, mg/l	pH	DO, mg/l	Cl$_2$, mg/l	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
1/28/73*	5	20	6	7	4.4	2.0	6.7					
	6	15	4	6	4.4	1.0	6.6					
1/29/73*	5	10	4	-	6.5	2.5	6.9					
	6	10	3	-	6.5	2.5	6.8					
1/30/73*	5	7	5	2	8.9	3.0	6.9					
	6	7	5	2	6.3	3.0	7.0					
1/31/73*	5	12	5	8	6.5	3.0	7.1					
	6	6	2	4	6.5	3.0	7.1					
2/1/73*	5	8	6	6	5.7	2.5	7.1					
	6	8	5	-	5.7	2.5	7.1					
2/2/73*	5	10	5	13	4.9	7.0	7.1					
	6	10	5	11	4.0	6.5	7.1					
2/3/73*	5	7	2	9	6.1	3.0	7.0					
	6	7	2	8	5.6	3.0	7.0					
2/4/73*	5	8	5	8	5.4	3.0	6.8					
	6	8	5	8	5.4	3.0	6.8					
2/5/73*	4	222	80		0.6	4.0	7.0					
	5	11	5	27	0.4	3.5	7.0					
	6	11	4	7	0.2	3.0	7.2					
2/6/73*	5	11	6	5	0.7	2.0	7.0					
	6	9	6	4	0.7	2.0	7.0					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
2/8/73*	5	45	17	41	6.1	6.0	7.0					
	6	45	17	35	6.1	5.5	7.0					
2/9/73*	5	33	11	41	1.0	2.5	7.0					
	6	33	11	40	1.0	2.5	7.0					
2/10/73*	5	127	44	12	1.8	1.6	6.8					
	6	106	22	4	1.3	1.6	6.9					
2/11/73*	5	71	30	35	7.8	7.5	6.7					
	6	32	30	35	7.8	7.5	6.7					
2/12/73*	5	84	27	40	8.1	7.5	6.8					
	6	84	27	40		7.5	6.9					
2/13/73	5	80	24	31		6.0	6.9					
	6	66	17	30		6.0	7.0					
2/14/73*	5	34	10	8		3.0	7.0					
	6	20	10	5		3.0	6.9					
2/15/73*	5	6	4	8	7.5	3.0	7.0					
	6	6	4	6	7.5	3.0	6.9					
2/16/73*	5	6	3	7	3.0	3.0	6.9					
	6	6	2	7	2.5	3.0	7.0					
2/17/73*	5	12	5	4	3.0	3.0	6.9					
	6	12	3	4	2.5	3.0	7.0					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
2/18/73*	5	24	4	8	1.7	2.5	6.7					
	6	24	4	3	1.7	2.5	6.8					
2/19/73*	5	20	4	4	0.3	2.5	7.0					
	6	16	4	2	0.3	2.5	7.0					
2/20/73*	5	17	5	6	1.6	0.1	6.9					
	6	14	5	2	1.5	0.1	6.9					
2/21/73	5	16	6	7	1.8	2.5	7.0					
	6	16		7	1.8	2.5	7.0					
2/22/73	5	175	59	48	3.8	13.5	6.9					
	6	86	44	38	3.7	13.0	6.9					
2/23/73	5	101	45	42	3.2	17.5	6.9					
	6	81	36	37	3.2	17.5	6.9					
2/24/73	4	411	166	188	9.8	24.0	7.0					
	5	122	55	58	3.6	13.5	6.8					
	6	107	44	48	3.6	13.5	6.8					
	8	29	13	4	2.9	14.5	7.0	9.5	4.0			36
2/25/73	5	92	42	48	4.4	13.5	6.9					
	6	92	42	41	4.2	13.5	6.9					
2/26/73	5	67	37	41	4.3	13.5	6.9					
	6	67	37	38	4.0	13.0	6.9					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	pH	DO, mg/l	Cl_2, mg/l	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
2/27/73	5	42	24	36	3.7	14.0	6.9					
	6	42	24	33	3.7	14.0	6.9					
2/28/73*	5	18	8	12	2.7	5.5	7.0					
	6	10	4	10	2.2	5.0	7.0					
3/1/73	5	66	34	46	2.6	11.5	6.9					
	6	49	26	43	2.5	11.5	6.9					
3/2/73	5	128	47	43	4.3	12.5	7.0					
	6	115	41	29	4.0	12.5	7.0					
3/3/73	5	138	42	32	5.4	13.0	7.0					
	6	132	42	29	4.9	12.0	7.0					
3/4/73	5	83	30	24	5.6	12.0	6.9					
	6	71	24	24	5.5	12.0	6.9					
3/5/73	4	678	212	66	4.9	3.0	7.0					
3/6/73	4	74	41	34	2.8	3.5	7.0					
	5	166	34	18	3.1	3.5	7.0					
	6	150	34	18	3.0	3.5	7.0					
	7							9.5	0.1	0	0	43
	8	30	9	4	2.5	6.0	7.0	9.1	0.2	0	0	41
3/7/73	4	560	156	55	4.2	3.5	6.8					
	5	133	28	18	2.8	4.0	6.8					
	6	127	28	15	2.6	4.0	6.8					
	7							7.2	0.1	0	0	41

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
3/8/73	5	99	16	14	3.1	3.0	6.8					
	6	95	16	13	2.8	2.5	6.8					
	7							8.2	3.0	0	0	44
	8	50	6	3	3.1	5.5	7.0			0	0	
3/9/73	5	97	25	5	2.6	5.0	7.0					
	6	88	25	5	2.6	5.0	7.0					
3/10/73	4	556	168	70	3.8	1.5	7.1					
	5	94	16	12	2.7	4.0	6.9					
	6	89	11	8	2.7	4.0	6.9					
	7							9.0	1.5	280	0	54
	8	54	10	2	3.0	6.0	7.0			0	0	54
3/11/73	4	208	56	49	2.0	0.2	7.2					
	5	205	40	8	2.1	2.5	7.0					
	6	175	32	8	2.1	2.0	6.9					
	7								2.0			
3/12/73	5	96	10	8	1.7	2.0	7.1					
	6	80	7	8	1.7	2.0	7.0					
3/13/73	5	178	26	7	1.8	11.5	7.1					
	6	154	22	6	1.4	11.5	7.2					
	7							9.5	1.0	0	0	48
	8	38	2	1	2.5	4.0	6.9	9.8		0	0	46
3/14/73	5	146	22	11	1.8	6.0	7.2					
	6	143	22	11	1.4	6.0	7.3					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./100ml	Fecal Coli./100ml	Temp., °F
3/15/73	5	172	26	8	2.4	3.0	7.3					
	6	172	26	8	1.8	3.0	7.3					
	7							10.1	1.0	0	0	48
	8	52	8	3	2.8	7.0	6.9	9.8		0	0	49
3/16/73	5	215	22	7	1.8	2.5	7.1					
	6	215	22	7	1.8	2.5	7.1					
3/17/73	5	288	28	14	2.0	1.8	7.0					
	6	262	24	9	2.0	1.4	7.1					
3/18/73	5	286	34	13	2.4	2.0	6.9					
	6	286	34	12	2.3	2.0	7.1					
3/19/73	5	144	20	11	1.1	2.0	7.0					
	6	144	15	11	1.1	2.0	7.0					
	7							10.3	1.0			38
	8	59	12	7	2.0	4.0	7.0		0.1			37
3/20/73	5	135	14	13	3.0	3.5	7.0					
	6	135	14	13	2.8	3.5	7.0					
3/21/73	7							12.0	2.0			35
	8	52	8	9	3.1	4.5	7.0					35
3/22/73	4	329	155	32	4.0	2.0	6.9					
3/23/73	5	70	12	21	3.1	3.0	6.9					
	6	70	12	20	2.8	3.0	6.9					
	7							9.0	0.1	0	0	36
	8	41	12	14	1.8	4.0	6.9	9.5			11	35

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/l	VSS, mg/l	BOD_5, mg/l	PO_4-P, mg/l	NH_3-N, mg/l	pH	DO, mg/l	Cl_2, mg/l	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
3/24/73	5	124	27	9	2.0	3.5	7.0					
	6	97	18	9	2.0	3.5	7.0					
3/25/73	5	110	101	9	2.0	6.0	6.9					
	6	23	22	9	1.5	6.0	6.9					
	7							10.0	0.1			44
	8	34	12	4	2.1	3.0	7.0	9.4	0.5			42
3/26/73	5	121	27	10	2.2	5.5	6.9					
	6	92	16	10	2.1	5.0	6.8					
	7							10.2	0.2			45
	8	26	6	3	1.4	4.0	6.9		0.1			43
3/27/73*	5	63	12	7	1.8	4.0	6.9					
	6	48	8	3	1.4	4.0	6.9					
3/28/73*	5	16	1	4	1.8	7.0	6.9					
	6	14	1	2	1.8	7.0	6.9					
3/29/73*	5	8	8	8	2.0	4.0	6.9					
	6	8	8	8	1.9	4.0	6.9					
3/30/73*	5			-	2.0	6.0	6.9					
	6	8	4	-	2.0	6.0	6.9					
3/31/73	5	38	13	23	4.4	2.5	7.0					
	6	17	6	21	4.3	2.5	7.0					

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
4/1/73	4	106	36	58	2.3	3.0	7.0					
	5	36	17	15	1.6	2.5	7.0					
	6	17	13	13	1.4	2.5	7.0					
	7							8.6	0.1	2280	17	51
	8	19	7	7	1.8	5.5	7.0			170	4	50
4/2/73	5	58	18	15	2.5	6.0	7.0					
	6	41	6	15	2.0	6.0	7.0					
4/3/73	5	86	16	11	2.5	3.0	7.1					
	6	47	6	8	2.3	3.0	7.1					
4/4/73	5	124	16	13	2.4	2.5	7.1					
	6	124	14	13	2.4	2.5	7.1					
	7							10.2	2.0			47
	8	16	3	6	2.0	2.5	7.0	8.2	0.1			45
4/5/73*	5	78	10	9	2.5	4.0	7.0					
	6	78	10	7	2.5	4.0	7.0					
4/6/73*	5	78	13	7	2.1	6.0	7.1					
	6	46	6	7	2.0	6.0	7.1					
	7							9.8	1.5			48
	8	20	8	3	1.8	6.8	7.0	8.5	0.1			48
4/7/73*	5	59	20	11	2.6	4.0	6.8					
	6	35	12	10	2.4	4.0	6.8					
4/8/73*	5	50	20	11	2.3	2.5	6.9					
	6	31	18	8	2.0	2.5	6.9					
	7								0.2			47
	8	18	11	4	2.4	3.0	6.9	9.5	0.1			46

TABLE B-1 (continued). POST-CONSTRUCTION STUDIES SAMPLING DATA

Sampling Date, mo/day/yr	Sampling Location	SS, mg/1	VSS, mg/1	BOD_5, mg/1	PO_4-P, mg/1	NH_3-N, mg/1	pH	DO, mg/1	Cl_2, mg/1	Total Coli./ 100ml	Fecal Coli./ 100ml	Temp., °F
4/9/73*	5	33	12	6	1.2	2.5	6.9					
	6	23	10	6	1.2	2.5	7.0					
4/10/73*	5	25	15	9	1.7	6.0	6.9					
	6	25	15	9	1.7	6.0	6.9					
	7											
	8	20	8	3	1.0	6.5	6.9	10.6	0.1			44 43
4/11/73*	5	25	10	5	2.3	2.5	7.0					
	6	17	10	5	2.0	2.5	7.0					
4/12/73*	5	19	15	6	2.0	9.0	6.9					
	6	15	14	5	1.8	9.0	6.9					
4/13/73*	5	16	11	7	2.1	2.0	6.9					
	6	16	10	5	1.8	2.0	7.0					
4/14/73*	5	8	6	9	1.6	3.0	7.0					
	6	6	5	8	1.6	2.5	7.0					
4/15/73*	5	18	15	11	1.7	2.0	6.8					
	6	6	5	11	1.4	2.0	6.8					
	7											
	8	10	8	9	2.2	3.5	6.8		2.0	0 0	0 0	51 50

TABLE B-2. POST CONSTRUCTION STUDIES OPERATIONS

Period		Unit operations											Data				Remarks
Date beginning, mo/day/yr	Date ending, mo/day/yr	Lake 1 aerators, N.	Lake 1 aerators, S.	P.S. #1	Microstr.	Chlorine	$NaClO_2$	L. 3 Aer.	P.S. #2	Filter	To river	To Lake 2	Lake 1 depth range, ft.	P.S. #1 pumpage, gpm	Cl_2 usage, lbs/day	P.S. #2 pumpage, gpm	
7/20/72	7/27/72		X	X	X	X		X	X	X	X		9-12	595	95	490	7/20/72-Power failure at site 7/21/72-Lakelets filled to capacity first time
7/28/72	8/1/72		X	X	X	X		X					8-10	510	80		
8/2/72		X	X	X	X	X		X					8-9	550	80		
8/3/72	8/13/72		X	X	X	X		X					6-11	550	87		8/3/72-Stormwater pump station sampler failure
8/14/72	8/15/72	X		X	X	X		X					5-12	550	82		
8/16/72	8/22/72		X	X	X	X		X	X	X	X		7-11	550	81	550	
8/23/72	8/31/72	X	X	X	X	X		X					8-9	550	85		
9/1/72	9/6/72		X	X	X	X		X					8-9	550	85		
9/7/72	9/11/72		X	X	X	X		X	X	X	X		8-9	720	85	550	
9/12/72	9/17/72		X	X	X	X		X					8-9	710	80		
9/18/72	9/21/72		X	X	X	X		X	X	X	X		7-8	600	82		
9/22/72	10/5/72												0				
10/6/72	10/22/72		X	X	X	X		X					0-10	550	65		Repair slope failure between Lakelet Nos. 1 & 2.
10/23/72	10/24/72		X	X	X	X			X	X	X		10-12	600	90	600	
10/25/72	10/28/72		X	X	X	X		X	X	X	X		10-13	600	90	500	
10/29/72	11/1/72			X	X	X		X					12-13	600	95		
11/2/72	11/5/72	X	X	X	X	X		X	X	X	X		13-14	500	90	500	

TABLE B-2 (continued). POST-CONSTRUCTION STUDIES OPERATIONS

Period		Unit operations											Data				Remarks
Date beginning, mo/day/yr	Date ending, mo/day/yr	Lake 1 aerators, N.	Lake 1 aerators, S.	P.S. #1	Microstr.	Chlorine	$NaClO_2$	L. 3 Aer.	P.S. #2	Filter	To river	To Lake 2	Lake 1 depth range, ft.	P.S. #1 pumpage, gpm	Cl_2 usage, lbs/day	P.S. #2 pumpage, gpm	
11/6/72	11/9/72	X	X	X	X	X		X					12-13	600	90		11/9/72-Sand filter breakdown (butterfly valve shattered)
11/10/72	11/15/72	X	X	X				X					11-13				
11/16/72		X		X		X		X					13	90			11/14/72-Sand filter back in operation
11/17/72	11/21/72		X	X	X	X		X	X	X	X		9-13	380	75	500	
11/22/72	11/23/72		X	X	X	X		X					8-9	410	60		
11/24/72	11/26/72				X	X		X	X			X	6-9		60	410	Recirculation
11/27/72	11/28/72	X		X	X	X		X					11	400	45		
11/29/72	12/1/72		X	X		X		X					10-11	410	45		Microstrainer backwash pump failure
12/2/72	12/6/72		X	X	X	X		X					7-9	260	45		
12/7/72	12/9/72		X	X				X	X	X	X						Lakelet No. 1 level recorder breakdown
12/10/72	12/12/72							X	X			X					Recirculation
12/13/72	12/17/72	X		X	X	X		X					6-11	300	50		
12/18/72	12/20/72			X	X	X							8-10	580	50		12/18/72-Lakelet No. 3 aerator capsized
12/21/72	12/25/72	X		X	X								7-9	500			
12/26/72	12/30/72				X	X			X			X			50	500	
12/31/72			X	X	X	X								500	50		Lakelet No. 1 full
1/1/73	1/2/73			X	X	X			X		X		8-9	500	50		

TABLE B-2 (continued). POST-CONSTRUCTION STUDIES OPERATIONS

Period		Unit operations											Data				Remarks
Date beginning, mo/day/yr	Date ending, mo/day/yr	Lake 1 aerators, N.	Lake 1 aerators, S.	P.S. #1	Microstr.	Chlorine	$NaClO_2$	L. 3 Aer.	P.S. #2	Filter	To river	To Lake 2	Lake 1 depth range, ft.	P.S. #1 pumpage, gpm	Cl_2 usage, lbs/day	P.S. #2 pumpage, gpm	
1/3/73	1/7/73			X	X	X			X	X	X		8-9	500	50	510	
1/8/73	1/9/73			X	X	X							8	500	50		
1/10/73					X	X			X			X	7		50	375	Recirculation
1/11/73	1/13/73			X	X	X							3-7	191	50		
1/14/73	2/5/73				X	X			X			X	3		50	350	Recirculation
2/6/73	2/7/73	X	X										3				
2/8/73	2/10/73	X	X		X	X			X			X	4		25	200	Recirculation
2/11/73	2/12/73				X	X			X			X	4		25	200	Lakelet No. 1 aerators frozen
2/13/73		X	X	X	X	X							4	200	25		
2/14/73		X	X		X	X			X			X	4		25	200	Recirculation
2/15/73	2/20/73				X	X			X			X	4		25	200	Lakelet No. 1 aerators frozen
2/21/73	2/23/73	X	X	X	X	X							4-6	200	25		
2/24/73	2/27/73		X	X	X	X							5	150	25		
2/28/73					X	X			X			X	4		25	150	Recirculation
3/1/73	3/2/73	X	X	X	X	X							5	170	25		
3/3/73	3/4/73		X	X	X								5	260			
3/5/73		X	X										5				

TABLE B-2 (continued). POST-CONSTRUCTION STUDIES OPERATIONS

Period		Unit operations											Data				Remarks
Date beginning, mo/day/yr	Date ending, mo/day/yr	Lake 1 aerators, N.	Lake 1 aerators, S.	P.S. #1	Microstr.	Chlorine	NaClO$_2$	L. 3 Aer.	P.S. #2	Filter	To river	To Lake 2	Lake 1 depth range, ft.	P.S. #1 pumpage, gpm	Cl$_2$ usage, lbs/day	P.S. #2 pumpage, gpm	
3/6/73	3/17/73		X	X	X	X							6-15	570	30		3/11/73-Lakelet No. 1 full
3/18/73																	Shut down
3/19/73	3/20/73		X	X	X	X							4	100	20		
3/21/73	3/22/73	X	X										4				
3/23/73	3/26/73		X	X	X	X								175	20		
3/27/73	3/30/73	X			X	X			X			X			20	400	Recirculation
4/1/73	4/4/73		X	X	X	X							3-9	500	20		
4/5/73		X	X		X	X			X			X	4		20	500	4/1/73 to 4/15/73-Recirculation, micro-strainer will not handle more than 500 gpm.
4/6/73	4/7/73				X	X			X			X	4		15	500	
4/8/73	4/9/73				X	X			X			X	4			500	
4/10/73					X	X			X			X	4		10	500	
4/11/73	4/15/73				X	X			X			X	4		50	500	

TABLE C-1. RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/l	VSS, mg/l	BOD_5, mg/l	pH	PO_4-P, mg/l	NH_3-N, mg/l	DO, mg/l	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
4/2/70	A	15	7	7	7.0	0.5	12.0	7.8	23		42
Storm	B	8	4	9	7.3	0.7	10.5	9.7	47		42
4/15/70	A	16	8	12	7.3	2.7	1.0	8.4	1		57
	B	14	6	8	7.5	2.9	2.0	9.6	6		57
5/5/70	A	28	16	16	7.1	1.2	0.5	8.4			64
	B	22	9	12	7.0	0.8	0.3	8.5			64
5/22/70	A	39	23	13	7.2	1.6	0.4	7.8	15		67
	B	21	7	18	7.2	1.2		8.0	24		67
6/10/70	A	22	15	14	6.9	0.8	1.2	7.7			73
	B	15	4	8	7.1	0.6	0.4	7.5			73
6/18/70	A	19	8	9	7.1	0.3	2.0	8.4	6		85
	B	21	10	11	7.1	0.9	0.5	8.2	1		85
7/2/70	A	43	16	11	6.9	0.4	1.6	4.2	12		78
	B	35	11	13	7.7	0.7	0.0	9.4	19		78
7/15/70	A	24	12	10	6.8	0.4	0.3	7.4			78
	B	36	17	16	7.1	0.9	0.4	7.7			78
7/28/70	A	26	10	8	8.0	0.2	0.5	8.5	10		79
	B	24	8	7	8.6	2.7	0.0	9.0			79

[a] Sampling location A = Up stream location
Sampling location B = Down stream location

TABLE C-1 (continued). RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/1	VSS, mg/1	BOD_5, mg/1	pH	PO_4-P, mg/1	NH_3-N, mg/1	DO, mg/1	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
7/29/70	A	19	5	13	7.7	0.5	1.5	8.0	26		82
Storm	B	200	81	13	6.5	4.9	10.5	8.5	14		82
7/31/70	A	47	9	10	7.1	4.7	0.6	7.2			84
Storm	B	22	9	11	6.9	3.4	2.0	7.8			84
8/11/70	A	43	20	12	6.8	0.4	0.5	6.7	21		84
	B	44	14	19	7.5	0.4	0.1	6.5	34		84
8/17/70	A	11	9	3	7.0	0.2	0.4	6.4	19		82
	B	15	12	17	7.2	0.5	0.2	6.1	15		82
8/23/70	A	22	10	6	7.3	0.6	0.3	6.0	12		83
Storm	B	23	9	6	7.6	0.3	0.3	6.2	17		83
8/29/70	A	14	9		7.8	0.4	2.5	6.9	10		78
Storm	B	10	8		7.8	0.5	3.5	6.7	6		78
8/31/70	A	14	6		8.0	0.7	4.5	6.7			79
	B	8	7		7.8	0.5	3.5	6.6			79
9/4/70	A	24	6	7	7.4	0.3	0.5	7.8	3	3	74
	B	26	6	4	7.5	0.2	0.5	7.6	12	17	74
9/10/70	A	34	7	6	6.6	2.3	4.3	7.0	1	7	70
	B	17	9	3	6.9	0.2	4.0	7.1	5	13	70

[a] Sampling location A = Up stream location
Sampling location B = Down stream location

TABLE C-1 (continued). RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/l	VSS, mg/l	BOD_5, mg/l	pH	PO_4-P, mg/l	NH_3-N, mg/l	DO, mg/l	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
9/16/70	A	102	64	14	6.0	2.0	10.2	7.2	12		70
	B	35	9	4	6.9	0.4	0.8	7.4	15		70
9/21/70 Storm	A	109	17	13	6.7	0.5	10.5	6.6	24	13	69
	B	103	16	13	6.8	0.5	14.0	6.7	38	17	69
9/23/70	A	60	11	11	6.6	6.6	10.0	6.8	32		70
	B			8	6.8	6.8	11.0	6.9	26		70
9/26/70	A			13	7.8	2.0	2.0	7.0	15	60	70
	B			15	8.4	3.5	3.5	7.2	24	130	70
9/29/70	A	106	26	12	7.6	0.6	6.0	6.9	14	4	70
	B	14	4	15	7.8	0.3	3.5	7.0	28	22	70
10/6/70	A			6	7.5	0.3	2.3	6.8	17	32	68
	B			11	7.7	0.5	2.6	7.0	21	14	68
10/12/70	A	40	18	12	6.8	0.2	1.0		14		57
	B	27	21	10	7.0	0.4	1.5		27		57
10/18/70	A	182	26	13	7.0	0.2	2.0		28		
	B	153	27	11	7.3	0.2	2.0		28		
10/20/70	A	31	21	10	6.8	0.3	0.5		11	9	64
	B	13	7	12	7.0	0.9	2.0		27	20	64

[a] Sampling location A = Up stream location
Sampling location B = Down stream location

TABLE C-1 (continued). RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/l	VSS, mg/l	BOD_5, mg/l	pH	PO_4-P, mg/l	NH_3-N, mg/l	DO, mg/l	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
10/24/70	A	78	25	12		0.2	0.5				
	B	33	19	10		0.6	1.0				
10/30/70	A	41	32	11	7.0	0.5	1.5		35		
	B	16	6	12	7.2	0.2	0.5		48		
1/3/70	A	34	17	12	7.0	0.3	1.0		21	14	
	B	60	36	13	6.9	0.9	10.5		34	23	
11/10/70	A	37	23	13	6.8	2.1	6.0		10	8	
	B	41	31	14	7.0	2.6	4.0		21	37	
11/19/70	A	67	33	10	7.0	0.4	6.0		2	6	
	B	57	37	10	7.0	0.2	3.5		7	18	
12/15/70	A	7	4	15	7.2	1.0	2.8		18	6	
	B	9	5	15	7.2	1.2	3.5		9	23	
4/1/71	A								3	14	
	B								0	0	
4/8/71	A	10	5	4	6.6	0.4	7.0	7.7	2	11	43
	B	8	4	6	6.9	0.5	6.0	7.7	2	11	43
4/12/71	A	17	4	4	7.3	0.1	1.0		4	12	52
	B	14	4	4	7.3	0.1	2.0		2	24	52

[a] Sampling location A = Up stream location
Sampling location B = Down stream location

TABLE C-1 (continued). RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/l	VSS, mg/l	BOD_5, mg/l	pH	PO_4-P, mg/l	NH_3-N, mg/l	DO, mg/l	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
4/16/71	A	16	4	4	7.3	0.4	4.0		7.8	23	50
	B	18	4	6	7.4	0.5	5.0		15.0	50	50
4/22/71	A	29	4	6	7.4	0.6	5.3	5.6	6.4	57	55
	B	21	6	10	7.3	0.7	5.6	5.4	8.2	76	55
4/28/71	A	12	5	4	7.3	1.0	14.5	7.6	6.6	106	50
	B	12	6	8	7.3	1.0	19.0	7.6	9.8	108	50
5/3/71	A	9	4	6	7.5	0.1	1.5	6.2	4.3	23	52
	B	12	6	16	7.2	0.6	4.3	3.2	5.1	46	52
5/10/71	A	34	21	20	7.2	2.1	3.5	6.0	6.7	78	62
	B	34	9	7	7.2	1.1	2.5	5.9	2.3	78	62
5/18/71	A	39	10	14	7.7	0.8	0.5	9.1	2.1	33	66
	B	27	12	9	8.1	0.2	0.5	9.8	0.1	4	66
5/20/71	A	20	10	8	6.9	1.0	2.0	0.5	1.7	17	66
	B	12	8	9	7.1	0.8	1.5	1.7	0.7	60	66
5/24/71	A	24	8		6.8	0.9	2.1	0.3	5.6	100	66
	B	16	8		7.0	0.6	1.0	1.7	1.0	10	66
6/2/71	A	19	3	14	6.8	0.8	6.5	7.5	8.0	121	62
	B	37	2	5	6.8	0.2	5.5	7.1	0.5	3	62

[a] Sampling location A = Up stream location
Sampling location B = Down stream location

TABLE C-1 (continued). RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/l	VSS, mg/l	BOD_5, mg/l	pH	PO_4-P, mg/l	NH_3-N, mg/l	DO, mg/l	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
6/7/71	A	24	10	4	7.8	0.6	6.0		4	10	73
	B	19	12	5	8.4	0.1	5.5		0.1	9	73
6/11/71	A	32	9	15	7.0	0.5	1.2	5.0	TNTC[b]	TNTC	68
	B	54	8	8	7.3	0.3	0.2	6.8	8	TNTC	68
6/18/71	A	17	11	6	6.2	0.7	2.5	5.6	TNTC	TNTC	73
	B	28	11	3	7.2	0.4	5.5	8.7	3	41	73
6/23/71	A	24	7	8	7.4	0.3	0.1	7.6	TNTC	TNTC	
	B	23	7	7	7.8	0.2		7.9	0.2	8	
6/30/71	A	28	9	11	7.2	0.3	0.2	7.6	4	TNTC	
	B	20	8	6	7.9	0.3	0.1	6.8	0.2	4	
7/14/71	A	12	7	3	6.7	0.4	0.4	5.7	TNTC	TNTC	
	B	30	8	3	6.9	0.2	0.1	6.5	0.5	23	
8/4/71	A	20	2	3	7.8	0.2	4.2	8.1	130	15	
	B	20	1	40	7.9	0.1	3.8	8.8		8	
8/10/71	A	19	2	4	7.3	0.1	1.1	8.1	300	42	
	B	34	2	3	7.4	0.1	1.0	7.9	300	6	
8/26/71	A	19	7		7.0	0.1	0.6	6.6	1200	110	
	B	24	4		7.4	0.1	0.3	7.2	130	28	

[a] Sampling location A = Up stream location
Sampling location B = Down stream location
[b] Too numerous to count

TABLE C-1 (continued). RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/l	VSS, mg/l	BOD_5, mg/l	pH	PO_4-P, mg/l	NH_3-N, mg/l	DO, mg/l	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
9/7/71	A	20	6		7.2	0.2	0.4	7.3	60	12	
	B	19	7	21	7.5	0.1	0.1	7.9	0	5	
9/16/71	A	25	9	4	7.5	1.1	0.1	6.2	70	13	
	B	39	12	4	7.8	0.8	0.1	6.8	40	10	
9/20/71	A	45	15	4	7.4	0.4	0.8	7.0	TNTC[b]	TNTC	
	B	33	7	3	7.6	0.1	1.0	7.3	380	20	
7/12/72	A	4	2		8.1	0.2	0.6	12.1	10	1	79
	B	5	2		8.4	0.2	0.6	11.0	40	2	79
7/13/72	A	19	8	7	8.0	0.1		10.1	50	3	76
	B	20	11	3	8.2	0.1		9.7	80	2	76
7/26/72	A	17	11	21	8.1	0.1	0.7	8.8	130	2	78
	B	17	16	18	8.1	0.1	0.5	7.4	180	6	78
7/27/72	A	26	9	16	8.2	0.4	0.4		40	70	79
	B	26	6	50	8.3	0.4	0.6		70	6	79
7/30/72	A	15	5		8.2	0.7		8.3	10	5	74
	B	18	4		8.4	1.0		8.7	30	3	74
8/10/72	A	10	6		7.8	0.1		6.6		3.0	
	B	8	4		7.9	0.1		6.7		4.0	

[a] Sampling location A = Up stream location
Sampling location B = Down stream location

[b] Too numerous to count

TABLE C-1 (continued). RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/l	VSS, mg/l	BOD_5, mg/l	pH	PO_4-P, mg/l	NH_3-N, mg/l	DO, mg/l	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
8/13/72	A	14	6	5	7.8	0.4	0.6	9.0	0	0	
	B	12	2	3	8.0	0.4	0.1	8.7	0.2	4.0	
8/14/72	A	9	2	4	6.9	0.3	2.5	7.8	0.21	5.6	
	B	8	4	3	7.4	0.1	0.5	8.4	0.34	4.6	
8/24/72	A	72	10	7	7.3	0.2	0.8	4.0	34.0	304.0	
	B	62	8	5	7.3	0.1	0.8	3.9	17.2	468.0	
8/29/72	A	25	13	1	8.1	0.4	0.5	8.1	0.240	24.0	
	B	26	9	1	8.4	0.3	0.5	8.0	0.250	25.0	
8/31/72	A	19	6	3	7.6	1.0	0.8	9.0	0.520	51.0	
	B	16	4	3	7.8	1.1	0.7	9.3	0.740	62.0	
9/1/72	A	25	8	5						62.0	
	B	21	8	4					0.740	51.0	
9/4/72	A	25	5	17				6.9			
	B	26	1	16				6.7			
9/5/72	A								0.470	17.2	
	B								0.760	18.5	
9/7/72	A	17	4								
	B	22	5								

[a]Sampling location A = Up stream location
Sampling location B = Down stream location

TABLE C-1 (continued). RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/l	VSS, mg/l	BOD_5, mg/l	pH	PO_4-P, mg/l	NH_3-N, mg/l	DO, mg/l	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
9/8/72	A							7.0	0.870		62
	B							7.6	0	0	62
9/11/72	A	25	3	8	7.3	1.0	3.8	3.3	0.250	11.5	69
	B	18	4	6	7.5	0.8	3.8	3.3	0.160	11.0	69
9/13/72	A	11	2	3	7.0	2.8	0.5	7.8	0.360	10.2	70
	B	2	2	3	7.5	1.3	0.4	8.4	0.040	3.8	
9/15/72	A	41	12	18	6.9		4.4	2.1		41.6	69
	B	28	3	7	7.1		3.6	1.7		45.6	
9/16/72	A	32	7	18	6.7	4.7	2.2	2.5		38.4	70
	B	55	10	7	6.9	6.5	2.0	2.6		80.8	
9/18/72	A	19	7	8	7.5	4.0	1.6			28.4	71
	B	28	10	8	7.2	3.1	1.0		0.500		
9/20/72	A	37	7	3	7.9	1.0	1.0			36.0	70
	B	30	5		7.7	0.3	0.4			8.5	
9/26/72	A	29	9	3	7.0	0.7	0.8	6.2			68
	B	51	11		7.2	0.2	0.1		1.890		
9/29/72	A	56	12	2	7.0	0.4	0.6	6.4	0	0	64
	B	26	5	1	7.2	0.2	0.1	6.6	0.200	11.0	64

[a] Sampling location A = Up stream location
Sampling location B = Down stream location

TABLE C-1 (continued). RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/l	VSS, mg/l	BOD_5, mg/l	pH	PO_4-P, mg/l	NH_3-N, mg/l	DO, mg/l	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
10/2/72	A	18	4	2	7.1	0.3	1.4	6.6	0	0	62
	B	29	6	2	7.2	0.2	0.3	6.8		4.0	62
10/6/72	A	21	3		6.2	0.2	0.2	6.2		15.6	62
	B	18	2		6.8	0.1	0.1	6.1		16.0	62
10/7/72	A	11	5	4	7.4	0.2	0.1	7.0		75.0	60
	B	26	7	6	7.3	0.4	0	7.3			60
10/11/72	A	19	4	29	7.3	1.8	0.5	7.4			59
	B	21	5	16	7.3	0.8	0.1	7.7			59
10/17/72	A	12	1	2	6.4	0.5	9.0	7.8			51
	B	22	3	1	6.8	0.3	9.0	8.2			51
10/20/72	A	6	1	2	6.4	0.5	1.0	9.0			48
	B	5	0	1	6.9	0.2	2.5	9.5			48
1C/21/72	A			4	7.1		1.5	9.0		5.4	49
	B			3	7.4		3.5	9.4		8.9	49
10/24/72	A	24	3	4	7.1	0.6	1.0	7.8			52
	B	34	3	4	6.4	0.5	2.0	8.7			52
10/29/72	A	23	9	3	7.6	0.4	0.2	7.4			50
	B	30	9	4	7.7	0.4	0.2	7.7			50

[a] Sampling location A = Up stream location
Sampling location B = Down stream location

TABLE C-1 (continued). RIVER SAMPLING DATA

Sampling date, mo/day/yr	Sampling location[a]	SS, mg/l	VSS, mg/l	BOD$_5$, mg/l	pH	PO$_4$-P, mg/l	NH$_3$-N, mg/l	DO, mg/l	Total coli. x10^3/ 100ml	Fecal coli. x10/ 100ml	Temp., °F
11/1/72	A B	8 10	4 2	3 3		0.4 0.4	2.1 2.5	8.3 7.8			50 50
11/3/72	A B	6	4	2 3	6.8 6.8	2.7 2.3	0.5 0.6			TNTC[b] TNTC	
11/6/72	A B	12 16	2 3	6 4	7.4 7.4	4.4 3.5	2.0 1.0	5.7 6.1	TNTC TNTC	TNTC TNTC	50 50
11/8/72	A B	27 10	10 11	4 11		0.7 0.8	2.1 2.5	6.6 2.6	TNTC TNTC	TNTC TNTC	52 52
11/13/72	A B	12 16	2 4	3 3	6.1 6.1	0.6 0.6	2.0 1.5	8.1 8.1	TNTC TNTC	87.0 TNTC	44 44
11/15/72	A B	20 20	3 3	7 6	6.5 6.5	0.3 0.3	2.0 3.0	9.0 9.2	TNTC TNTC	TNTC TNTC	40 40
11/18/72	A B	10 4	4 2	22 5	6.8 6.8	0.4 0.5	0.7 1.2	8.3 8.5	TNTC	TNTC	43 43
11/20/72	A B	16 14	4 6	2 3	6.8 6.8	0.2 0.3	1.3 1.2	8.8 9.0	TNTC TNTC	TNTC TNTC	42 42
11/27/72	A B	20 24	2 5	3 4	6.9 6.9	0.1 0.5	1.1 0.4	8.6 9.0	12.600 15.800	TNTC TNTC	40 40

[a] Sampling location A = Up stream location
Sampling location B = Down stream location

[b] Too numerous to count

TABLE C-2. PRE/POST-CONSTRUCTION STUDIES RAINFALL DATA
(4/4/70-9/28/71 & 7/20/72-4/15/73)

Date mo/day/yr	Precip., inches	Date, mo/day/yr	Precip., inches
4/1/70	0.32	6/2/70	0.10
4/2/70	1.02	6/3/70	0.03
4/13/70	0.05	6/4/70	T
4/19/70	0.40	6/5/70	T
4/20/70	0.37	6/6/70	T
4/21/70	0.23	6/11/70	0.65
4/24/70	0.09	6/14/70	0.07
5/1/70	0.15	6/15/70	T
5/4/70	T	6/17/70	1.51
5/6/70	T	6/18/70	0.45
5/8/70	0.03	6/21/70	0.05
5/12/70	0.25	6/22/70	T
5/13/70	0.25	6/24/70	0.37
5/14/70	0.69	6/26/70	0.30
5/15/70	0.65	6/27/70	T
5/16/70	0.10	6/28/70	0.08
5/22/70	0.15	6/29/70	0.05
5/23/70	0.16	6/30/70	T
5/24/70	0.42	7/3/70	0.12
5/25/70	0.48	7/4/70	0.08
5/31/70	0.03	7/8/70	0.36
6/1/70	0.06	7/9/70	0.04

TABLE C-2 (continued). PRE/POST-CONSTRUCTION STUDIES RAINFALL DATA (4/4/70-9/28/71 & 7/20/72-4/15/73)

Date mo/day/yr	Precip., inches	Date, mo/day/yr	Precip., inches
7/10/70	0.07	9/15/70	0.37
7/12/70	0.30	9/17/70	0.84
7/13/70	0.04	9/18/70	0.50
7/14/70	0.19	9/22/70	0.21
7/19/70	0.38	9/23/70	0.04
7/20/70	0.27	9/26/70	0.77
7/23/70	0.08	9/28/70	0.01
7/28/70	0.03	9/30/70	T
7/29/70	0.72	10/10/70	0.07
7/30/70	0.15	10/12/70	0.17
7/31/70	0.26	10/13/70	0.15
8/2/70	0.01	10/14/70	0.29
8/3/70	0.09	10/20/70	0.36
8/19/70	0.72	10/21/70	0.04
8/22/70	0.16	10/28/70	0.01
8/29/70	0.35	10/29/70	0.21
8/30/70	0.07	10/30/70	0.02
9/3/70	0.09	11/1/70	0.16
9/5/70	T	11/2/70	0.56
9/6/70	0.04	11/3/70	0.28
9/13/70	0.01	11/4/70	0.18
9/14/70	0.12	11/5/70	T

TABLE C-2 (continued). PRE/POST-CONSTRUCTION STUDIES RAINFALL DATA
(4/4/70-9/28/71 & 7/20/72-4/15/73)

Date mo/day/yr	Precip., inches	Date, mo/day/yr	Precip., inches
11/6/70	T	12/22/70	0.07
11/9/70	0.14	12/23/70	0.06
11/10/70	0.04	12/25/70	0.03
11/14/70	0.24	12/26/70	0.07
11/15/70	0.01	12/30/70	T
11/20/70	0.53	1/3/71	0.16
11/21/70	T	1/4/71	0.11
11/22/70	T	1/17/71	0.03
11/27/70	0.36	1/22/71	0.03
11/28/70	0.18	1/26/71	0.07
11/29/70	0.16	1/29/71	0.10
12/3/70	0.28	2/3/71	0.05
12/4/70	T	2/4/71	1.06
12/5/70	T	2/5/71	0.29
12/6/70	T	2/11/71	0.07
12/10/70	0.06	2/12/71	0.16
12/11/70	0.55	2/17/71	0.21
12/12/70	0.12	2/18/71	0.11
12/13/70	0.08	2/19/71	0.48
12/16/70	0.23	2/20/71	0.03
12/17/70	0.02	2/22/71	0.51
12/19/70	0.03	2/23/71	0.05

TABLE C-2 (continued). PRE/POST-CONSTRUCTION STUDIES RAINFALL DATA
(4/4/70-9/28/71 & 7/20/72-4/15/73)

Date mo/day/yr	Precip., inches	Date, mo/day/yr	Precip., inches
2/26/71	0.02	4/27/71	0.09
2/27/71	0.03	5/1/71	0.05
3/4/71	T	5/5/71	0.04
3/5/71	0.04	5/12/71	0.01
3/6/71	0.40	5/19/71	0.35
3/7/71	0.13	5/24/71	0.25
3/10/71	0.20	5/25/71	0.02
3/11/71	0.08	6/1/71	0.23
3/15/71	0.03	6/2/71	0.23
3/18/71	0.05	6/4/71	T
3/19/71	0.20	6/6/71	0.10
3/20/71	0.01	6/13/71	0.26
3/21/71	0.01	6/19/71	T
3/22/71	0.09	6/20/71	0.01
3/23/71	0.10	6/22/71	T
4/1/71	0.16	6/26/71	0.12
4/4/71	T	7/5/71	0.74
4/12/71	0.03	7/17/71	0.04
4/13/71	0.14	7/19/71	T
4/17/71	0.09	7/22/71	T
4/19/71	T	7/26/71	0.22
4/22/71	T	7/30/71	0.88

TABLE C-2 (continued). PRE/POST-CONSTRUCTION STUDIES RAINFALL DATA (4/4/70-9/28/71 & 7/20/72-4/15/73)

Date mo/day/yr	Precip., inches	Date, mo/day/yr	Precip., inches
7/31/71	0.01	9/26/71	0.10
8/3/71	T	9/27/71	0.41
8/4/71	T		
8/10/71	0.19	CONSTRUCTION PERIOD	
8/14/71	T		
8/19/71	T	7/18/72	1.69
8/21/71	0.02	7/19/72	T
8/22/71	0.20	7/21/72	0.35
8/24/71	T	7/23/72	0.01
8/25/71	0.05	7/26/72	0.10
8/26/71	1.34	7/27/72	0.12
8/27/71	0.31	8/1/72	0.58
9/4/71	0.22	8/2/72	0.65
9/6/71	0.15	8/6/72	0.03
9/11/71	T	8/7/72	0.15
9/14/71	T	8/8/72	0.23
9/19/71	T	8/13/72	T
9/20/71	0.81	8/14/72	1.03
9/21/71	T	8/16/72	0.09
9/22/71	T	8/17/72	0.02
9/24/71	T	8/19/72	0.03
9/25/71	0.06	8/22/72	0.35

TABLE C-2 (continued). PRE/POST-CONSTRUCTION STUDIES RAINFALL DATA (4/4/70-9/28/71 & 7/20/72-4/15/73)

Date mo/day/yr	Precip., inches	Date, mo/day/yr	Precip., inches
8/23/72	0.09	10/31/72	T
8/24/72	0.16	11/1/72	0.05
8/26/72	0.20	11/2/72	0.40
9/2/72	0.20	11/3/72	0.01
9/8/72	1.13	11/4/72	0.01
9/11/72	0.03	11/7/72	0.56
9/13/72	0.71	11/8/72	0.14
9/17/72	0.03	11/11/72	0.03
9/18/72	0.19	11/13/72	0.41
9/24/72	0.05	11/14/72	0.44
9/25/72	0.11	11/15/72	0.03
9/26/72	0.29	11/19/72	0.19
9/27/72	T	11/20/72	0.06
9/29/72	0.38	11/25/72	0.26
9/30/72	0.20	11/26/72	0.38
10/15/72	0.02	11/29/72	T
10/16/72	0.01	12/1/72	0.08
10/21/72	0.44	12/4/72	0.16
10/22/72	1.27	12/6/72	0.51
10/23/72	0.59	12/8/72	0.18
10/28/72	0.42	12/10/72	0.01
10/29/72	0.17	12/12/72	1.21

TABLE C-2 (continued). PRE/POST-CONSTRUCTION STUDIES RAINFALL DATA
(4/4/70-9/28/71 & 7/20/72-4/15/73)

Date mo/day/yr	Precip., inches	Date, mo/day/yr	Precip., inches
12/13/72	0.03	2/22/73	0.02
12/15/72	0.10	2/23/73	0.01
12/19/72	0.05	2/25/73	0.05
12/20/72	0.03	2/26/73	0.03
12/26/72	0.09	3/5/73	0.51
12/29/72	0.36	3/7/73	0.14
12/30/72	0.42	3/9/73	0.04
1/3/73	0.64	3/10/73	0.02
1/22/73	0.63	3/11/73	0.83
1/23/73	0.01	3/14/73	1.11
1/24/73	T	3/16/73	0.10
1/28/73	0.06	3/17/73	0.80
1/29/73	T	3/28/73	0.05
1/31/73	T	3/29/73	0.54
2/1/73	0.09	3/31/73	0.21
2/2/73	0.35	4/1/73	0.19
2/7/73	0.13	4/2/73	0.03
2/8/73	0.04	4/9/73	0.36
2/14/73	0.03		
2/15/73	0.09		
2/20/73	0.02		
2/21/73	0.17		